CHAPTER 1 CHARACTERIZATION OF REGIONAL SEISMICITY AND GROUND SHAKING HAZARDS

Introduction

The objective of this chapter is to introduce the seismic and geologic hazards that have contributed to severe damage at port facilities during recent earthquakes, as well as provide an overview of the methods that are commonly used in practice to evaluate these hazards. This chapter is intended to serve as a resource document and as such, the introductory material is supplemented with numerous references to assist interested readers in locating additional sources of practical information on hazard analyses for ports. In addition to the references that address the specific topics discussed herein, the reader is directed to several recent publications which provide comprehensive treatment of seismic and geologic hazards (e.g., Kramer, 1996; Okamoto, 1984) and seismic design issues for port facilities (e.g., Port of Los Angeles, 1990; Tsinker, 1997). The recent report "Guidelines for Evaluating and Mitigating Seismic Hazards in California" prepared by the CDMG (1997) is a particularly worthwhile reference.

A comprehensive seismic hazard evaluation addresses topics such as the spatial and temporal occurrence of earthquakes, the characteristics of the ground motions that may be anticipated over specified time intervals, and the dynamic response of soils subjected to the design level ground motions. These factors collectively define the seismic hazard at a site. The evaluation of these hazards will generally proceed in five primary steps that include: (a) identifying potentially active seismic sources in the region, (b) estimating the seismicity associated with the individual sources, (c) evaluating the influence that the travel path has on the characteristics of the seismic waves as they propagate from the source to the site, (d) assessing the dynamic response of near surface soils (addressed herein), and finally (e) analyses of the stability of foundation soils and structures subjected to the design level ground motions. The collection and synthesis of this information involves input from geology, seismology, and engineering disciplines.

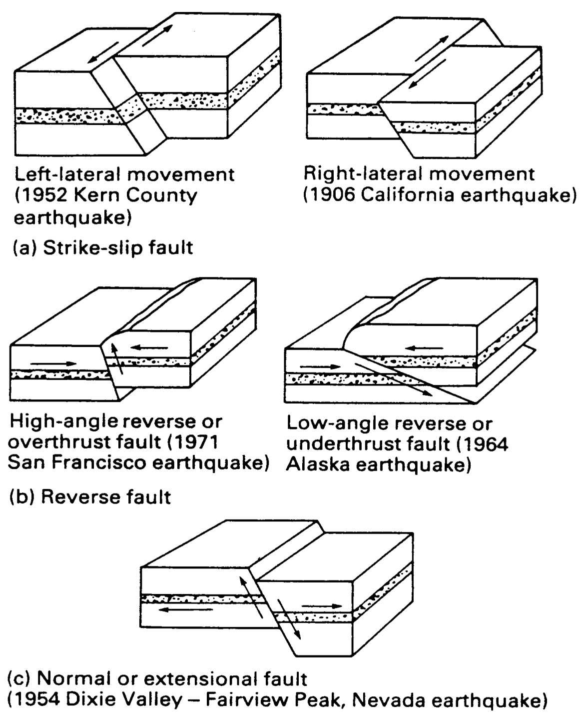
Seismic Source Identification and Characterization

The identification of seismic sources in the region of interest and the evaluation of the seismicity attributed to these sources forms the basis for the seismic hazard analysis. Primary seismological issues include: the location of local earthquake sources, the rate of seismicity attributed to each source as a function of earthquake magnitude, and the maximum credible earthquakes that could be generated by each seismic source. This evaluation requires the synthesis of geographic, geologic, and seismological data. An outline of the major components of a seismic hazard evaluation is provided in the following sections.

Characterization of the Regional Tectonic Setting

Earthquakes are associated with the release of strain energy along discontinuities in the earth's crust. These discontinuities, or faults, are manifested as crustal plate boundaries and fractures within plates. Most of the world's seismicity occurs along the plate boundaries due to the relative motions of the adjacent plates. Significant earthquakes have, however, also occurred within the crustal plates (termed *intraplate* earthquakes) along ancient rift zones and in regions of volcanic activity. The style of faulting, rate of seismicity, and the size of the greatest earthquakes associated with a potential seismic source are related to the tectonic processes in that region and the resulting stress patterns in the shallow crust. Excellent introductions to the seismological aspects of seismic hazards (i.e., global tectonics, the causes of earthquakes, earthquake magnitude scales and related topics) have been presented by Bolt (1993) and Idriss (1985).

The progressive increase in crustal stresses leads to the failure of crustal rocks along faults. The direction of the fault rupture is used to characterize various fault types. Three general types of fault - strike-slip, reverse, and normal - are illustrated in Figure 1-1. Pure examples of these fault types rarely occur; rather, the relative movement along the fault has components both parallel and normal to the fault trace. It is important to characterize the pattern of crustal stresses in that this determines the type and depth of faulting, and influences hazards such as surface faulting, enhancement of ground motions due to the rupture mechanism, and directivity of potential seismic energy release.



b) Fault types

Figure 1-1: Fault Types (Werner, 1991)

A variety of tectonic regimes exist in California. The tectonic provinces in the United States can be generally classified based on the mechanisms that produce the earthquakes and the style of faulting that is observed. For example, (a) subduction zones found in the coastal regions of northernmost California (north of Cape Mendocino) lead to thrust-type earthquakes (e.g., M_s 7.0 1992 Cape Mendocino earthquakes); (b) transform faulting along the western margins of California produce predominantly strike-slip earthquakes (e.g., M_s 8.2 1906 San Francisco earthquake, M_w 6.9 1989 Loma Prieta earthquake); (c) intraplate rift zones and Basin and Range faulting have produced large historic earthquakes in the eastern portion of California; and (d) volcanic processes have also produced significant earthquakes. With respect to marine oil terminals located in coastal regions and along inland waterways the seismic hazard is associated with the first two types of faulting.

Local variations in the crustal stress fields and fault geometry within broad regional tectonic regimes can result in earthquakes of different rupture process in the same region (e.g., M_s 6.4 San Fernando and M_w 6.7 1994 Northridge thrust-type earthquakes). In this the symbols M_s and M_w denote the surface wave magnitude and the moment magnitude of the earthquakes, respectively. These represent two of several magnitude scales that have been developed by seismologists (Bolt, 1993; Idriss, 1985). The surface wave magnitude and moment magnitude are the most commonly referenced magnitude scales in engineering practice and the difference in the magnitudes is minor in the range of practical interest (M 6 to 8).

The identification of regional faults, fault systems, or seismic source zones is the first step in the seismic hazard evaluation. The location of seismic sources is based on contributions from historic observations, surface mapping of offset strata, surface geomorphology, trenching studies, geophysical imaging, aerial photo interpretation, remote sensing, and geodetic leveling.

With respect to the historic record of earthquakes, the US Geological Survey National Earthquake Information Center, Denver, Colorado has produced EPIC, "The Global Hypocenter Data Base" (NEIC, 1992), a CDROM which contains parameters for more than 438,000 earthquake events. Seven world-wide and 12 regional earthquake catalogs were assembled to produce this data base, spanning a time period from 2100 BC through 1990. Useful data for the United States is generally constrained to the period when instruments were available to compute event magnitude. Each earthquake is detailed where data are available with date, origin time, location, magnitude estimates, intensity, and cultural effects.

A computer program, EPIC, is available for searching the CDROM. EPIC makes data available to information users via a user-defined search request. The request determines which steps are necessary to produce the desired output. An automated plotting package that produces seismicity maps in multicolor or monochrome is incorporated into the EPIC software. The data to be mapped are extracted from the selected data and plotted in a global or regional format. The availability of the CDROM data base of epicenters and EPIC software greatly facilitates creation of the historical epicenter subset required for use with automated site seismicity analysis tools. Details are presented in the EPIC user's manual, which will not be repeated here.

A number of data fields for some events are unfilled because the information is not available. Information on cultural effects, intensity, and other phenomena associated with the event has been included for earthquakes in the United States. The quality of epicenter determinations varies significantly with the time period studied. Before 1900, locations are usually noninstrumentally determined and are given as the center of the macroseismic effects. Most instrumental epicenters prior to 1961, excluding local earthquakes in California, were located to the nearest 1/4 or 1/2 degree of latitude and longitude. Reliable information on the quality of many epicenter determinations is lacking. Beginning in 1960, epicenters have been determined by computer, and the accuracy is generally better. However, although stated to tenths or hundredths of a degree, the location accuracy is usually a few tenths of a degree. Since May 1968, the latitude and longitude values for most events have been listed to three decimal places. This precision is not intended to reflect the accuracy of the location of events except for local California earthquakes and special epicenter determinations. Where several sources have determined an epicenter for the same earthquake, one solution has been designated as the most reliable. Usually it is the source believed to contain the best data set for the earthquake. In some cases, data from two sources were combined to provide a more complete record. Magnitudes from a number of different sources are included in the earthquake data file. Gutenberg and Richter (1954) and Richter (1958) discuss the development of the magnitude scale. Many magnitudes published by Gutenberg and Richter (1954) were later revised by Richter (1958). The revised magnitudes are used in the file even though the source is identified as Gutenberg and Richter (1954). The concept of earthquake magnitude is not restricted to one value. Several definitions are possible, depending on which seismic waves are measured. Three different magnitude scales, body wave (m_b), surface wave (M_s), and local (M_l), are distinguished in this file. In addition, another data field, other_magnitude , was included when it was unclear which scale was used. Recent earthquakes are being defined by moment magnitude. Richter (1958) and other modern seismology references provide detailed discussions on the topic of magnitude determination. The different scales do not give exactly comparable results, and different values frequently are given for the same earthquake (Idriss, 1985). It is common practice to average the individual magnitudes from different stations to get a more uniform value within each scale.

In general, the file contains earthquakes of magnitude less than 4.0 only for the United States region and for areas within dense seismic station networks. However, no claim is made for the statistical homogeneity of these events. Inclusion of earthquakes of magnitude 4.0 to 5.0 also is influenced by the proximity of seismic stations to the source or epicenter. A maximum intensity is listed for many of the earthquakes. Each is assigned according to the Modified Mercalli Intensity Scale of 1931 (Bolt, 1993). Some of these values have been converted from reported intensities on other scales.

A period of demonstrated quiescence over a geological time period indicates inactivity of the fault and probable continued inactivity. However, inactivity over a period of historic recording (50 to 100 years) does not imply future inactivity. Rather, it may point to a region which is locked and through which a major fault rupture may propagate. A number of earthquakes producing damage in southern California occurred on faults lacking historic activity. Caution must be exercised to recognize that the limitations of an incomplete data base when extrapolating to return periods greatly exceeding the length of the period of recorded data. Furthermore, aftershocks must be distinguished from main shocks. An area having recently undergone a large event releasing strain built up for hundreds or thousands of years is probably safe against a large release in the near future. Thus, a recent large event on a fault might actually indicate safety in the immediate future, rather than an indication of increased activity. A single event by itself cannot give an accurate measure of return time.

Despite the tremendous advances in fault identification that have been made possible by geophysical imaging studies, field mapping, and deep drilling projects, uncertainties still exist in the identification of faults capable of generating damaging earthquakes. Faults may lack surface expression due to burial under thick sedimentary deposits, or the combination of very low deformation rates and active geologic processes such as erosion which obliterate evidence of faulting. Several notable examples of concealed seismic sources include: “blind” thrust faults (M_w 6.7 1994 Northridge earthquake), rupture along folded strata at depth (M_s 6.7 1983 Coalinga, CA earthquake). Of particular concern may be the existence of potentially active off-shore seismic sources that have not been well characterized. Once faults have been identified, the seismicity associated with the feature must be assessed (as discussed below). Detailed maps that identify active faults are available in only a few areas of the United States. One such region is the San Francisco Bay area in California, Figure 1-2. The network of faults associated with the San Andreas Fault System are relatively well defined in this region, and distances between the site of interest and the local faults are well constrained.

In several regions of California the tectonic processes responsible for historic earthquakes have not been well defined. This is particularly true in regions of diffuse and low level seismicity such as the Central Valley (Sacramento-San Joaquin). Although these regions have been characterized as exhibiting relatively low seismicity, notable earthquakes have occurred (e.g., MMI IX 1892 Vacaville earthquake, MMI IX 1892 Winters earthquake)). In these regions the tectonic provinces are established from the geologic history of the region, structural trends in geologic units, geographic terrain, and measured crustal movements from geodetic investigations.

Evaluation of Potential Seismic Sources

The second step in a seismic hazard analysis incorporates historic seismicity, geologic evidence for prehistoric earthquake activity (termed *paleoseismicity*), and comparisons with similar tectonic regions around the world in order to establish the seismicity of the regional seismic sources. At this stage of evaluation the rate of seismicity (i.e., the recurrence interval for probable, earthquake are established. The historic record has been earthquakes of various magnitudes) and the estimation of the maximum credible, or used as one of the primary indicators

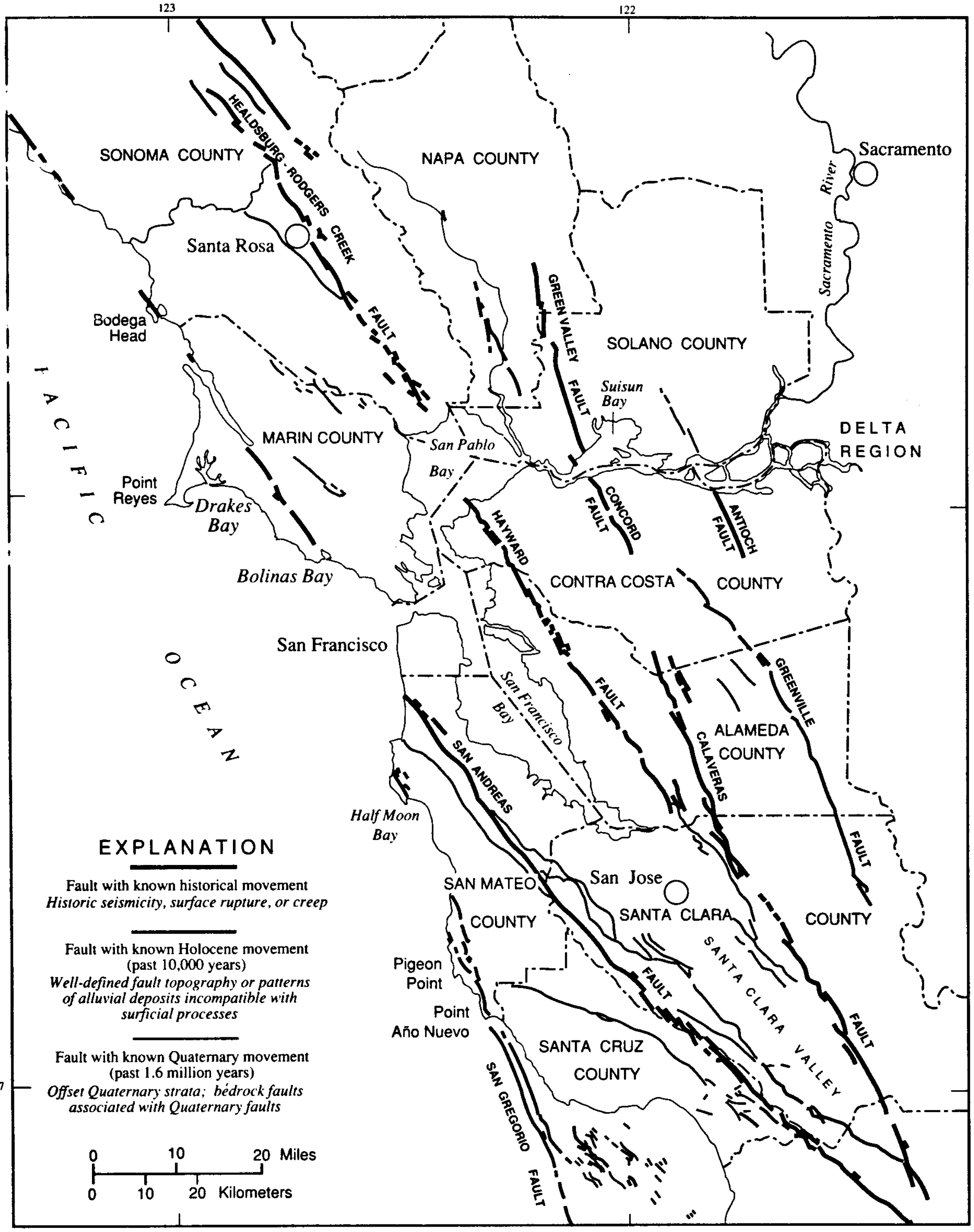


Figure 1-2: **Faults In The San Francisco Bay Region (Modified From Brown And Kockelman, 1983)**

of earthquake activity in most regions of the United States. The plate boundaries are well defined along the western United States by the historical patterns of seismicity. It is evident that although the highest rates of seismicity are found along coastal California, potentially damaging earthquakes have occurred during the historic record in many regions of the state. Several of these occurred in coastal regions and along inland waterways in other portions of the state.

The historic record of earthquakes in most of the United States is relatively short when compared to the length of time that would be required to accurately assess, from a statistical standpoint, the "average" rate of seismicity in a region. For this reason the historic record of earthquakes should be viewed as an incomplete indicator of seismicity levels in a region. In this situation the historic record of earthquakes must be supplemented with indirect lines of geologic evidence for faulting. Geologic investigations include: geomorphology studies of ground surface features along faults (e.g., sag ponds, offset streams, rift valleys), fault trenching studies, paleoseismicity investigations which look for evidence of ground failures caused by prehistoric earthquakes, and geophysical investigations to detect the deformation of soil and rock units at depth. From an engineering perspective, a fault is considered "active" if evidence has been found for earthquakes during the Holocene Epoch (i.e., the last 11,000 years).

The hazard posed by a potential seismic source is directly related to both the size of the earthquakes generated along the fault and the recurrence interval for damaging earthquakes. Given the relatively short historic record of earthquakes in most regions of the United States, it is doubtful that the "true" pattern of seismicity has been established in most regions. As a result of the scarcity of long-term seismicity data, the maximum credible earthquake (MCE) that can be attributed to a potential seismic source is usually specified independent of the time (i.e., deterministically) on the basis of geometry of the fault and the tectonic setting. The rate of seismicity is, by definition, based on the recurrence intervals between earthquakes of various magnitudes. Establishing the recurrence intervals for earthquakes will reflect the historic rate of seismicity, as well as the length of the historic record of earthquakes. In most regions of the United States, the combination of a relatively short period of observation (200 to 400 years) and low to moderate seismicity requires that the rate of seismicity used as the basis for engineering design be based on probabilistic methods of analysis. Useful overviews on the characterization of seismic sources have been prepared by Cluff and Coppersmith (1990), Coppersmith (1991) and Power et al. (1986).

Maximum Earthquake Magnitude

Once the potential seismic sources in a region have been identified, the maximum earthquake magnitude is estimated from historical seismicity and/or geologic data. Methods which are used to estimate the largest earthquake that may be generated by a specific source without regard to the length of time that may elapse between earthquakes of this size are termed deterministic. By specifying the MCE as independent of time, the worst case scenario is established. Empirical techniques which relate fault geometry to the magnitude of the largest earthquake are commonly used in deterministic analyses. One such method which relates measured surface rupture lengths to the moment magnitudes of the causative earthquakes is shown in Figure 1-3. It is common for practitioners to use relationships such as this to estimate the maximum credible earthquake for faults based on the mapped length of the fault. For example, this technique could be applied to the faults illustrated in Figure 1-2. While this provides a simple estimate for the maximum earthquake along a fault several points should be noted. First, experience has demonstrated the faults rarely rupture from end to end during a single earthquake, therefore the use of the entire mapped fault length could yield overconservative magnitude estimates. Equally important is the fact that fault segments that may or may not appear to be connected based on geologic information can rupture together during one earthquake (as in the M_s 7.5 1994 Landers, CA earthquake). The effect of this segmentation is apparent in Figure 1-2 where significantly different magnitude estimates are obtained if different fault segments are assumed to be connected (e.g., Healdsburg-Rogers Creek fault

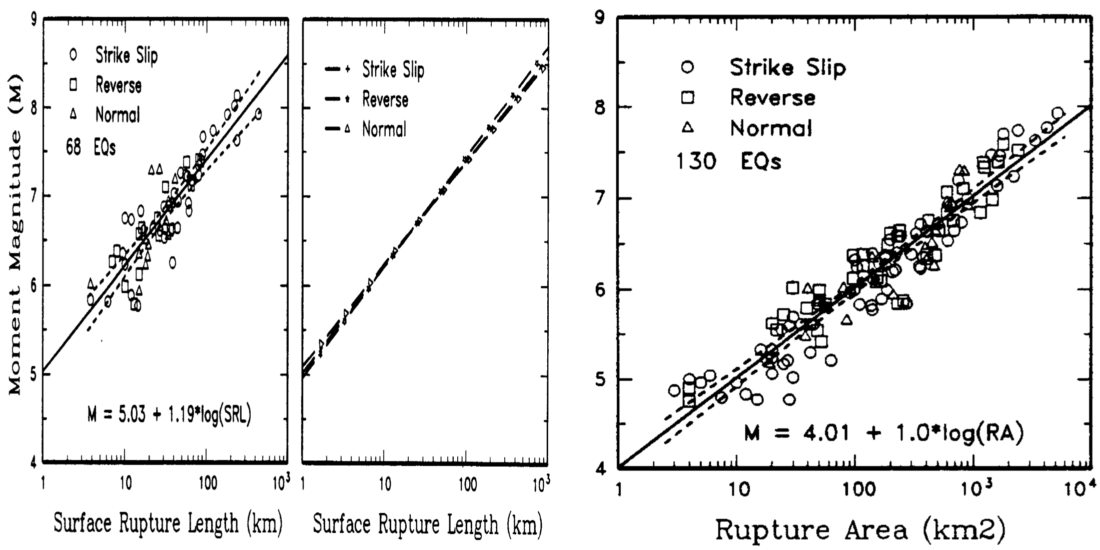


Figure 1-3: Relationships Between The Rupture Length And Earthquake Magnitude (Wells And Coppersmith, 1994)

and the Hayward fault; the Green Valley, Concord, and Calaveras faults). Additionally, this method of evaluation also assumes that the actual length of the fault has been identified.

Earthquake Recurrence Rates

Seismicity rates used in hazard analyses are estimated using two primary methods; magnitude-frequency of occurrence relationships based on historical seismicity, and recurrence intervals based on the slip-rates along active faults.

Historical Seismicity Estimates for the recurrence intervals for potentially damaging earthquakes are established from historic seismicity data and, to a lesser extent, on geologic evidence for prehistoric earthquakes. In many regions the historic record is sufficient to define the seismic hazard associated with small to moderate sized earthquakes (M 4 to 6). The uncertainty increases with increasing magnitude (which are usually the magnitudes of interest for engineering purposes). Seismicity data can be plotted as the number of earthquakes that have occurred (as functions of the duration of the historic record and the area of the region considered) versus magnitude in the form shown in Figure 1-4. The equation for the line on this semi-logarithmic plot is termed the *Gutenberg-Richter equation* and it has the form:

$$\log_{10}N = a - bM \qquad 1\text{-}1$$

where *N* is the number of shocks per year for a given magnitude (M), 10^a is the mean yearly number of earthquakes of magnitude greater than or equal to zero, and *b* describes the relative likelihood of large and small earthquakes. Mathematically, the b-value is the slope of the logN versus M line and it is widely used to model regional seismicity rates.

The b-value is important in that it represents the rate of seismicity for the region. Significant uncertainty can exist in specifying the b-value since the line is not well constrained for small magnitudes due to limitations in earthquake detection, and more importantly, for the larger magnitudes due to the incomplete record of earthquakes and the relatively small number of large earthquakes. The plot is, however, very useful in demonstrating that as the specified return period, or “exposure”, increases the size of the largest earthquake that is likely to occur during that span of time also increases. This increase in magnitude is not a linear function of time. Furthermore, the anticipated earthquake magnitude does not continue to increase as the recurrence interval increases. The maximum earthquake will be limited to the maximum credible earthquake established using deterministic methods of evaluation.

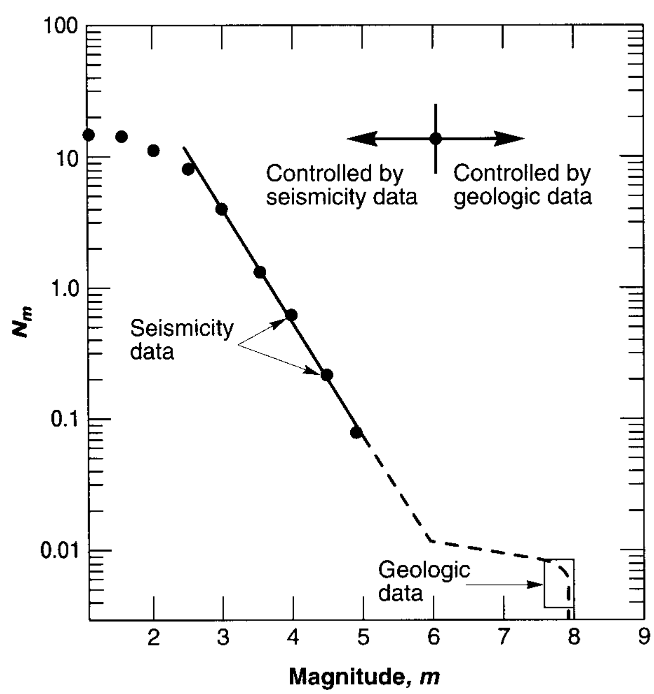


Figure 1-4: Inconsistency Of Mean Annual Rate Of Exceedance As Determined From Seismicity Data And Geologic Data (Youngs And Coppersmith, 1985)

Geologic Data It is very difficult to establish the Gutenberg-Richter frequency-magnitude relationship for individual faults. Given this shortcoming, an alternative method that is based on the average slip rate along the fault is used to supplement the historic record of earthquakes for an individual source. Slip rates obtained from geomorphology studies, fault trenching, geophysical investigations, or estimates made by comparisons to other faults in similar tectonic regimes are related to the moment magnitudes (proportional to the area of the fault rupture times the average fault displacement, or slip, during the earthquake) for credible earthquakes. Long term slip rates can be used to estimate the average rate of seismic moment release from a fault. Using relationships between earthquake magnitude and seismic moment one can then use the fault slip rate to estimate the rate of earthquake occurrence on the fault (Geomatrix, 1995).

Seismic hazards in a region are rarely due to a single seismic source. In most seismically active regions several faults or a network of closely spaced faults contribute to the overall seismic hazard. The maximum credible earthquake and seismicity rates will vary for each source. The seismic hazard at a port facility in these regions will reflect the combined hazard from each of the individual sources. In light of the fact that the recurrence intervals for damaging earthquakes generated along each fault vary, the hazard must incorporate the contribution of each of the faults for the return period of interest. The synthesis of the seismicity data for each source in the evaluation of the seismic hazard for a specific site is usually performed using probabilistic analysis methods.

Probabilistic seismic hazard analyses are commonly used to evaluate the relative contributions of each seismic source on the overall seismic hazard. Probabilistic methods offer several distinct advantages in establishing the contribution of the individual seismic sources on the aggregate hazard at a site. These include: (a) varying rates of seismicity can be incorporated, (b) the influence of uncertainties in the source characterization on the resulting design level motion can be evaluated, and (c) the influence of return period on the anticipated intensity of ground shaking can be determined. These factors are assessed through the use of “logic-trees” which allow systematic consideration of uncertainty in the values of the parameters of a particular seismic hazard model. Introductions to the concepts behind probabilistic analyses and their practical application are contained in Coppersmith (1991), Kramer (1996), Power et al. (1994), and the Working Group on California Earthquake Probabilities (1990).

The use of logic trees facilitates the incorporation of alternative hazard models in evaluating the seismic hazard at a site. As shown in Figure 1-5, each of the different models are assigned a weighting factor which indicates the relative reliability of the parameters being used in the model. The logic tree in Figure 1-5 was developed during a seismic source characterization study of the. The headings list various aspects of the fault geometry that affect the recurrence relationships for subduction earthquakes along the interface between the Juan de Fuca and North American plates. The weighting factors are given in parentheses and these represent the relative confidence that the investigators had in the individual models. It should be noted that the specific weighting values used are subjectively determined from empirical data on the characteristics of faults in similar tectonic environments and considerable judgment.

A Procedure For Computing Site Seismicity (U.S. Navy Method, Ferritto,1994)

As noted by Coppersmith (1991), many elements of seismic source characterization depend on the tectonic environment. A seismic model must be based on the knowledge of the local area. It can consist of either an area source zone or a detailed fault definition region. Where specific faults are identified as contributing to the regional seismic hazard these sources can be modeled as a line source extending along prescribed portions of the fault. Where a fault exhibits variations of activity along its length, it can be divided into subelements containing regions where activity is uniform.

For the procedures developed herein, a fault segment can be modeled by two line segments defined by three points. The events to include or associate with the fault are defined by specification of a distance from the fault line, such that all those events within the distance are grouped with the fault. Alternatively, a region can be designated by four points to bound the fault. Again, note a fault can be divided into pieces where activity or geometry so dictates.

Source zones are specified as regions where a zone of like seismicity is evident. The regional geology and tectonics assist in defining the source zone boundaries. A source zone is defined as a region of uniform seismicity, such that an event is equally likely to occur in any portion of the zone. This is characterized by the concept of a "floating earthquake," an event that can occur anywhere in the zone.

In the development of a site model, it is important to keep in mind that an equivalent representation of a region is being created by a series of fault line segments or source zones. The seismicity must be captured in terms of its spatial location and in terms of the level of activity. Assignment of events to one fault source as opposed to another increases that fault's contribution to the estimation of event recurrence. It is important to capture all of the regional the seismicity. For faulting conditions where there are a number of parallel elements, it may not be easy to separate which events are associated with which fault. Consideration must be given to the dip of the fault in assigning events, since the epicenter for a sloping fault can actually occur in a number of kilometers away from the surface trace of the fault. The large majority of strike slip faults have steep dips of 70 degrees or greater. On the other hand, thrust faults generally have dips much less than this, generally in the range of 45 to 60 degrees. For the cases where a fault is close to a site (within 10 miles), special considerations should be given to the location of the fault line segments that define the fault model. If the fault dips toward the site, the actual epicentral distance may be closer to the site than the surface trace of the fault. For faults at greater distances, the difference becomes less significant.

Once a fault or region has been defined as a seismic source, the maximum earthquake magnitude must be defined. In a previous section, a plot was shown relating fault rupture length to magnitude. The length of a fault can be estimated from maps. An assumption can be made that a fault will rupture over 50 to 80 percent of its length. This estimate of rupture distance can be used to define the fault magnitude. Estimates of fault magnitudes have been made for some Western United States faults. It is essential to review previous geologic and seismological studies for the region to develop an understanding of the site's tectonic setting and seismic potentials.

Computation Of Recurrence Parameters

The procedures discussed in this section are equally applicable to regional analysis or fault analysis. The subset of events assigned to the source zone of interest are used to calculate the Richter A and B coefficients, Equation 1-1 above. This computation defines the earthquake recurrence as a line on a semilog plot. The linear segment is bounded by a maximum magnitude determined as discussed above and by a minimum magnitude below which the data becomes nonlinear. Typically, the value of B is about -0.9. The general earthquake recurrence is thus initially defined. However, as will be shown in the following sections, two important elements are added to geologic slip data and characteristic magnitude.

Geologic Slip-Based Recurrence

A procedure were presented above for calculating recurrence based on the geologic slip rate data. Once the seismicity is estimated from the historical data, the geologic data can be compared. The procedure allows the user to adjust the A and B values from the historical data to include information based on the longer span geologic data. Should other studies be available, the results of these individual fault studies can be used here by adjusting the recurrence parameters.

Characteristic Magnitude

As discussed above, geologic data may show the presence of history of a characteristic event at some average return time. The seismicity defined by the historical data fails to capture this activity, so it is important to include it within the set of events developed for the fault. Once the size of the event and the effective average return time is defined, it is possible to include this in the analysis. Again, if studies with more advanced models are available to define temporal distributions, that data can be used here.

Computational Procedure

Various approaches were presented above to determine the probability of earthquake occurrence. As shown above, various amounts of data are required, some of which are beyond the scope of an engineering investigation. The engineer is free to use any documented procedure which will achieve valid results.

One approach was taken in the formulation of a Monte Carlo simulation procedure, Ferritto (1994). The procedure uses the fault model and regional model discussed earlier, together with the recurrence procedure. As stated above, the A and B parameters combined with geologic slip rate data and characteristic magnitude form the basis for the recurrence function. Once the recurrence function for a fault is defined, the magnitude distribution can be computed. The process is done for each fault individually. A list of 5,000 events representing the largest magnitudes expected to occur in 50,000 years is computed. For each magnitude, a fault break length is determined using data by Coppersmith (1991). A random epicenter location is selected along the fault. The fault break is then assigned to the random epicenter. Various distances are computed, such as epicentral distance, hypocentral distance, and closest distance of fault break to site. The choice of distance depends on the acceleration attenuation equation chosen by the user. Using the magnitude and separation distance, a site acceleration and standard deviation are computed. A random acceleration is then determined. Associated with each acceleration is the causative event and distance. The process is repeated 5,000 times for each fault. The random fault data are then combined for a total site probability distribution. The procedure described above has the advantage that historical data are augmented with available geologic slip data. Where characteristic events are defined, they may be easily incorporated at the appropriate return time. The effective nonlinear recurrence function attempts to capture the temporal characteristics of the data without complex estimates of Markov or Bayesian parameters.

Crustal Deformation Hazards

The process of fault rupture and the release of strain energy results in permanent crustal deformations. The surface manifestation of this deformation will reflect factors such as the type of faulting (thrust versus strike slip), the magnitude of the earthquake, and the nature of the near-surface rock or soil (Bray et al., 1994). These seismic hazards include relatively deep seated crustal deformation, and surficial deformations such as ground rupture, and creep along the fault. Deep seated deformations tend to be very broad and regional in nature while surficial deformations are generally very localized along faults. The seismic hazards associated with tectonic deformations can be generally confined to (a) regional vertical deformations in regions of

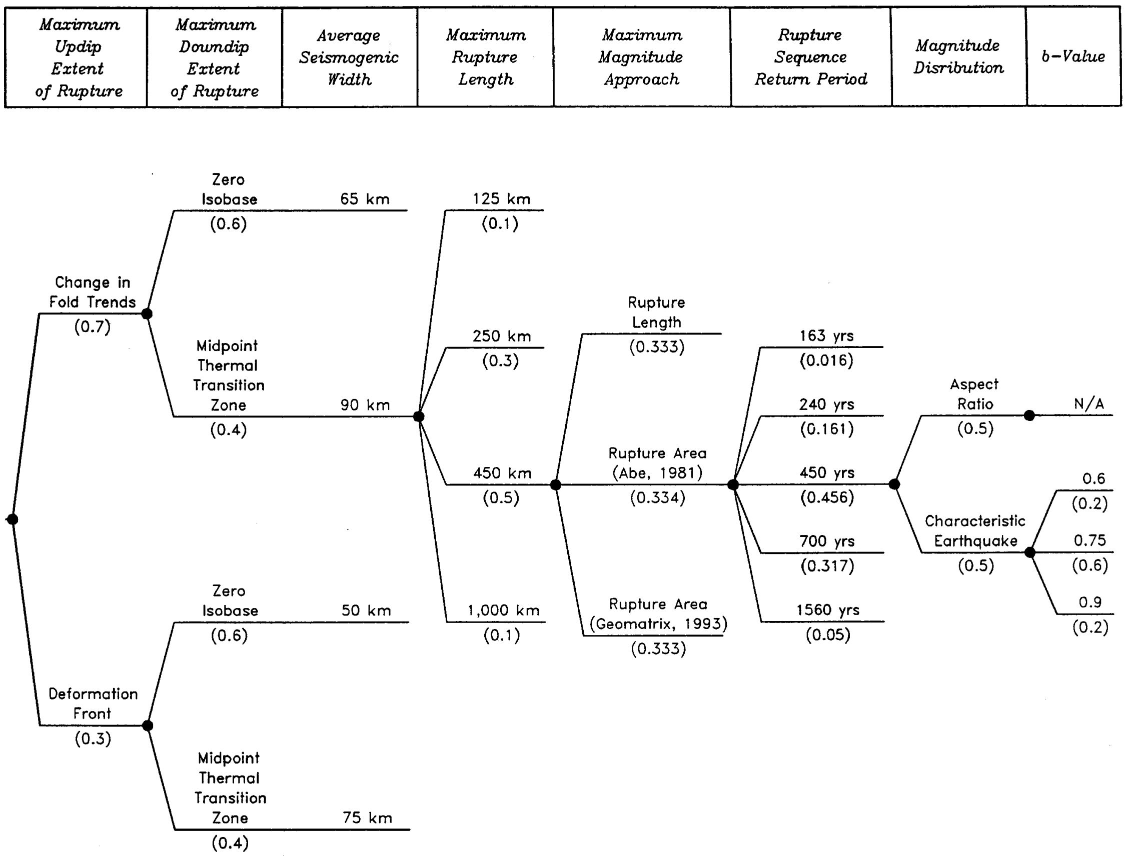


Figure 1-5: Seismic Source Characterization Logic Tree For Cascadia Interface Source (Geomatrix, 1995)

dip-slip faults (thrust or extensional faulting), and (b) surface deformations which occur along, or in close proximity, to faults that rupture at the ground surface. The first of these scenarios may affect regions with subduction zone earthquakes while the second phenomena will be of concern to ports that are located on active faults.

Ground Surface Rupture Ground surface rupture is commonly associated with earthquakes of magnitude 5.5 and greater. The extent of the rupture and the displacement across the fault generally increases with magnitude. Average displacements across the faults vary from approximately 1 cm for M_w 6.0 to as much as 7.5 meters for M_w 8. Experience during earthquakes demonstrates that lifelines (e.g., water, power, transportation and communication lines) that cross active faults are vulnerable to damage from fault offset. Although the likelihood of fault passing directly through a port is small, port authorities should be aware of the lifelines that serve the port, and the routing of these lifelines relative to local faults.

Creep Slow, aseismic crustal movements across faults are termed creep. Although this phenomenon is not associated with strong ground motions, it does constitute a seismic hazard in several regions of the United States. As with earthquake induced displacements across faults, long-term creep affects lifelines that cross the fault by producing relative offsets.

Ground Shaking Hazards

This section provides background information on earthquake ground motions, including the characterization of strong ground motions for engineering design purposes, the geological parameters that affect the strength of the ground shaking, and current methods for estimating site-specific ground motion parameters. The behavior of soil deposits and structures during earthquakes is dependent on the strength, frequency content, and duration of ground shaking. The strength and duration of the ground motions are fundamentally related to the seismic energy imparted into a body, while the frequency content is important in assessing the response of structures. Procedures for characterizing each of these parameters are summarized below.

Characterization of Ground Motions

For earthquake engineering of structures and soil materials, potential ground motions at a site are characterized in terms of their strength, frequency content, and duration. These characteristics are discussed below. Further discussion of these ground motion characteristics with regard to their use in the seismic design and analysis of port structures is provided in following chapters.

Strength of Earthquake Ground Motions

Qualitative Measures The strength of earthquake ground shaking has historically been characterized on the basis of qualitative intensity estimates and, with the advent of strong motion recording instruments, peak acceleration and other quantitative single-parameter measures of the strength of the shaking as obtained from recorded ground motions. The intensity of the ground motions is established using a qualitative scale that uses Roman numerals to represent the effect of the earthquake shaking; (a) on persons in the felt area (i.e., human perception of the ground motions); (b) the response of structures and other objects; and (c) the ground (i.e., geologic phenomena induced by the earthquake shaking). One such scale -- the Modified Mercalli Intensity scale -- has been widely used in North America as a method of representing the strength of earthquake motions in the absence of recorded motions. Other qualitative intensity scales are used in other parts of the world. Maps which illustrate the relative intensity levels in affected areas (termed isoseismal maps) have been used to estimate ground motion parameters for use in engineering analysis and design in regions lacking instrumental strong motion data (Bolt, 1993; Kramer 1996).

Quantitative Measures It is common in engineering design and analysis to characterize the strength of the ground shaking using simple, single-parameter, quantitative measures, such as peak acceleration, velocity, and displacement; and effective peak acceleration, velocity, and displacement). Such parameters have gained wide acceptance because they are easily incorporated into standard pseudostatic methods of analysis. However, the characterization of a transient time history of motion using a single peak ground motion amplitude fails to account for other important aspects of the motion (i.e., frequency content and duration). In order to overcome the deficiencies in peak ground motion parameters, parameters based on energy concepts and spectral response (as discussed below) have been developed (e.g., root-mean-square acceleration, power spectrum intensity, Arias intensity, response spectrum intensity). Descriptions of each of these parameters are found in Kramer (1996), Naeim (1989), and Naeim and Anderson (1993).

Frequency Characteristics of Ground Motions

To define the frequency content of the ground shaking, a response spectrum or Fourier amplitude spectrum is required. Of these, the response spectrum is most widely used in engineering practice, since it describes ground motion frequency characteristics in a form that is directly applicable to structural design and analysis. The ground response spectrum is obtained by applying the ground motions to the base of a suite of single-degree-of freedom oscillators all having equal damping ratios, and plotting the maximum response of the oscillator as a function of its natural frequency or natural period (Newmark and Hall, 1982; Kramer, 1996). This is depicted schematically in 1-6. This peak response can be plotted in either linear form using absolute acceleration and relative velocity and displacement, or in tripartite form using “pseudo”-accelerations, velocities and displacements (a simplified computational technique that relates each of the ground motion parameters by multiples of 2π /T). These plots are particularly useful for demonstrating the predominant period of the earthquake motions.

Numerous empirical relationships for estimating response spectra for ground motions have been developed during recent statistical regression analysis of available strong motion data (Seismological Research Letters, 1997). The results of most of these recent studies have been presented in the form of simple, straightforward equations that allow the engineer to calculate spectral ordinates at the period of interest as a function of parameters such as earthquake magnitude, source to site distance, type of faulting, site geology, etc.

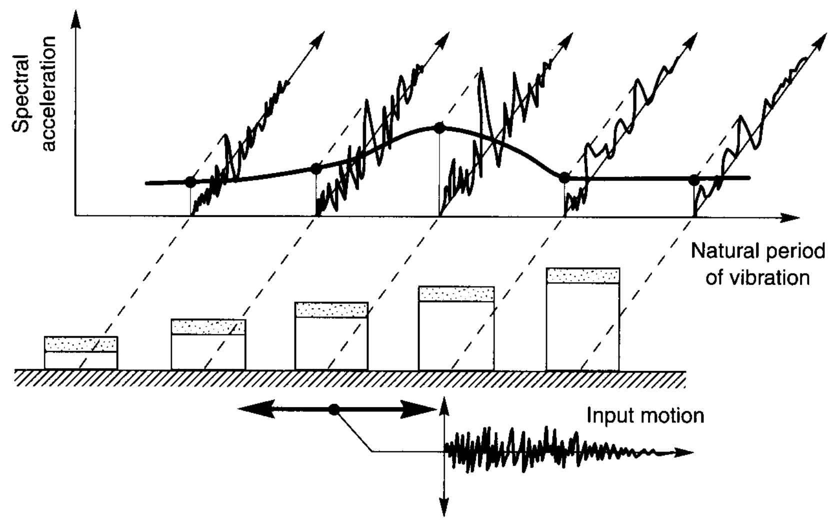


Figure 1-6: The Response Spectrum Obtained By Plotting The Spectral Accelerations Against The Periods Of Vibrations Of The S.D.O.F. Systems (Kramer, 1996)

As the seismic waves propagate from the rupture zone, the high frequency components of the motion are attenuated more quickly than lower frequency motions. This is due to damping by the transmitting rock which dissipates a fraction of the wave energy per cycle of travel. Since the high frequency waves have shorter wave lengths, they are attenuated more quickly with distance from the rupture than the longer period motions. The frequency dependent attenuation of ground motions results in a shift in the predominant period of the ground motions with increasing travel distance. The variation of predominant period at rock outcrops with magnitude and distance is shown in Figure 1-7.

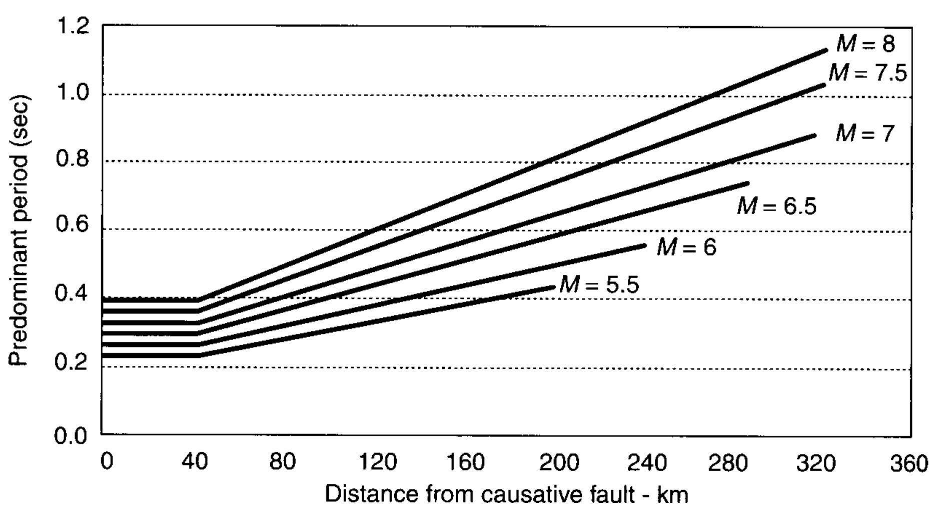


Figure 1-7: Variation Of Predominant Period At Rock Outcrops With Magnitude And Distance (After Seed Et Al., 1969)

Duration of Ground Motions

In addition to the strength and frequency content, the duration of the ground shaking will also influence the seismic performance of structures. This is particularly true for ductile structures designed to yield when subjected to strong ground motions. The inelastic response of such structures is sensitive to the number of cycles of strong motion that will be applied during the earthquake. The duration of shaking is also vital in the stability of cohesionless soils and performance of slopes and embankments.

The duration of strong shaking increases with increasing earthquake magnitude. The potential for earthquake-induced damage is a function of the duration of *significant* ground motions. For this reason, the concept of a “bracketed” duration has been used, which is defined as the length of time from the first exceedance of a specified acceleration level to the last exceedance of that acceleration level. Because the threshold for damaging motions is in the range of 0.05 g to 0.20 g for many structures, “bracketed” durations for acceleration levels of 0.05 g, 0.10 g and 0.20 g have been widely used (Naeim and Anderson, 1993). The variation of bracketed duration with magnitude and epicentral distance is shown in Figure 1-8.

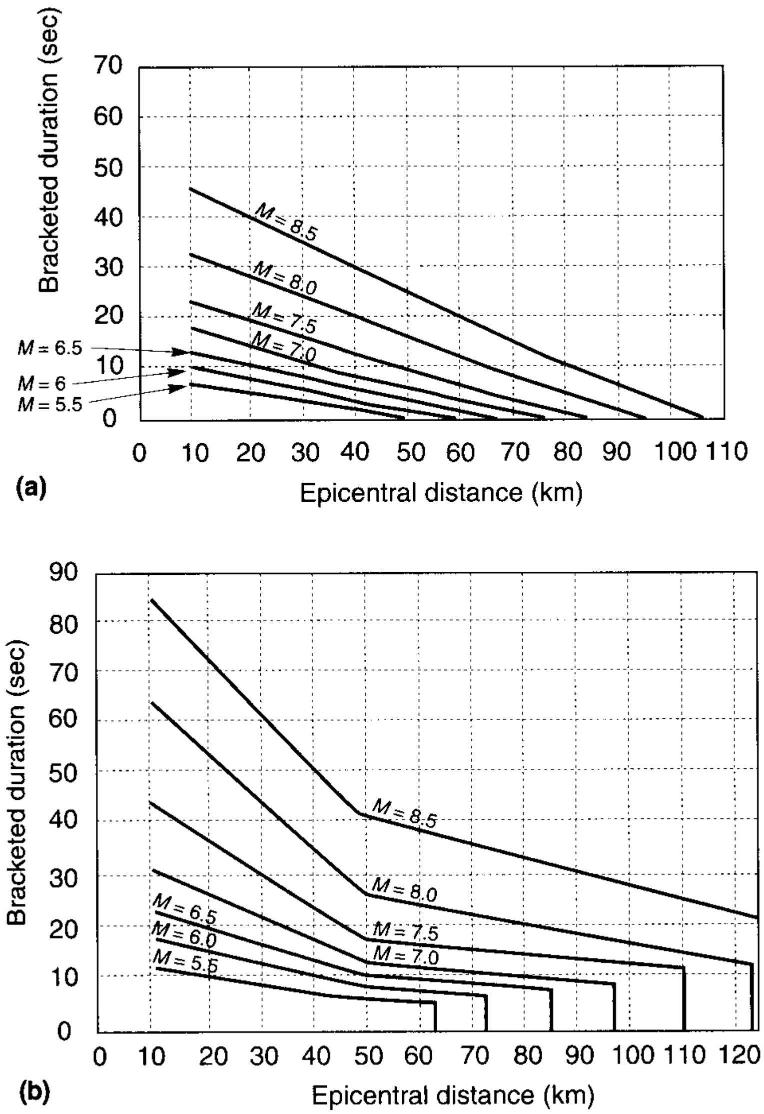


Figure 1-8: Variation Of Bracketed Duration (0.05 G Threshold) With Magnitude And Epicentral Distance: (A) Rock Sites; (B) Soil Sites (After Chang And Krinitzsky, 1977)

Factors Affecting Ground Shaking

The ground shaking characteristics at a particular site can be affected by numerous factors related to the fault rupture process, the propagation of the seismic waves as they travel from the ruptured fault to the site, and the local soil conditions at the site. These factors are briefly summarized below.

Fault Rupture Process The characteristics of the fault rupture that could influence the ground shaking at a site are the stress drop, the total fault displacement, the length of the fault break, the nature of the rupture process (i.e., the single or multiple fault breaks that can occur), the fault shape, and the proximity of the fault plane to the ground surface. In addition, whether the fault ruptures in a single direction or as bi-lateral rupture will significantly affect the duration of ground shaking. For example, both the 1989 Loma Prieta Earthquake (M_w = 6.9) and the 1995 Hyogoken Nanbu Earthquake (M_w = 6.8) featured bi-lateral rupture of the causative fault from the earthquake hypocenter. Because of this, the duration of the ground motions that were recorded during each of these earthquakes was much less than what might ordinarily be expected from earthquakes with the above magnitude levels. Although both earthquakes caused significant damage in the surrounding areas, these damage levels would undoubtedly have been much greater had the rupture been in a single direction rather than bi-lateral.

Travel Path Effects As seismic waves radiate away from the fault rupture zone during an earthquake the characteristics of the waves are modified. The strength of the ground shaking decreases due to geometric spreading of the wave front and damping of the waves as they propagate through the crustal rock. The frequency content of the motions is also affected by the dynamic behavior of the rock and the distance that the waves have traveled. The influence of the propagation path and transmission properties of the crustal rock on the seismic waves have been combined as "path effects." Once all potential seismic sources in the region of interest have been identified, the source-to-site distances can be scaled. Given the distance from the rupture to the site and a very general classification of the regional crustal rock, the path effects can be evaluated

As previously mentioned, geometric spreading and damping result in the attenuation of seismic waves. The decrease in the strength of the ground motions has been modeled numerically, although the most widely used attenuation relationships are based on statistical analyses of recorded ground motions. As regional arrays of strong motion instruments have become more common around the world, the data base of recorded motions is rapidly expanding. Statistical analyses of arrays of these recorded motions have been performed to develop a multitude of attenuation relationships for various regions, types of faulting, and site conditions. Recent overviews of this work are contained in the Seismological Research Letters (1997).

Many of the attenuation relations focus on the variation of peak acceleration or peak velocity with distance from the rupture zone. The example in Figure 1-9 shows the attenuation of peak acceleration and provides a comparison of several widely used empirical relationships for ground motions due to earthquakes in the western United States. Several factors must be considered when using empirical attenuation relationships:

A variety of distance measures have been used in establishing the attenuation relationships. Such measures include; (a) distance to the rupture plane, (b) distance to the vertical projection of the rupture plane, (c) epicentral distance, and (d) hypocentral distance.

The composition and integrity of crustal rocks will have a pronounced influence on the attenuation of ground motions. As illustrated in Figure 1-10, ground motions are felt over a much broader region in the eastern and central United States than in the western portion of the country. This is due to the age and composition of the rocks in the respective regions. In a general sense, the west coast is underlain by predominantly younger sedimentary rocks which have relatively high damping characteristics, while bedrock in the central and eastern regions is commonly much older metamorphic and igneous rock which has much lower damping and is much more efficient in transmitting the seismic waves. The empirical relationships are therefore only applicable to regions with roughly similar geology.

The type of faulting and the depth of the rupture have been shown to influence the rate of attenuation. This is particularly evident with subduction zone earthquakes. As a result, in regions such as northernmost California and the Pacific Northwest that are prone to subduction zone earthquakes, attenuation relationships developed specifically for such earthquakes are used when evaluating seismic hazards associated with these thrust-type earthquakes (Seismological Research Letters, 1997).

The near surface geology at the site is specified in several studies (hard rock, soft rock, shallow stiff soil, etc.).

For many years, very little strong motion data was available for source-to-site distances less than 15 km to 20 km. This was especially true for earthquake magnitudes in the range of engineering interest (M 6). However, since the mid-1980s, analytical studies and analyses of near-field recorded motions have indicated that the position of the site relative to the fault could influence the characteristics of the ground motions at the site. The analytical studies demonstrated that, for a moving source, waves that leave the traveling rupture zone in opposite directions will have different amplitudes, much like the Doppler effect in acoustics. Near-field strong motion recordings (e.g., from the 1989 Loma Prieta, 1994 Northridge, and 1995 Hyogoken Nanbu earthquakes) have demonstrated the significance of near source effects such as rupture directivity, or “fling”, for seismic design of structures located within about 10 km of the rupture zone.

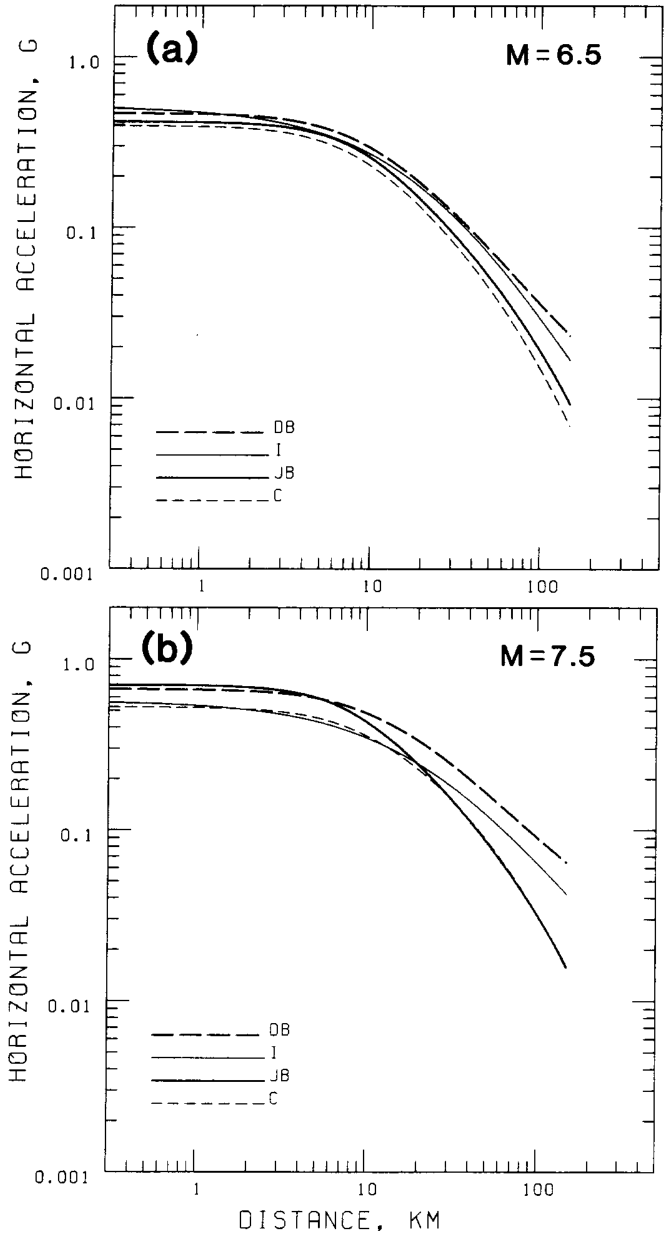


Figure 1-9: Comparison Of Different Relationships For Horizontal Acceleration At Magnitudes 6.5 (A) And 7.5 (B) (Joyner And Boore, 1988)

The propagation of the rupture toward a site at a velocity that is almost as large as the shear wave velocity of the rock causes most of the seismic energy from the rupture to arrive in a single large pulse of motion which occurs at the beginning of the record (Somerville et al., 1997). This large pulse results in enhanced near-field ground motions, particularly ground velocities and displacement components that effect longer period structures. The extent of this effect is quite variable, and depends on the azimuth of the site with respect to the direction of rupture. In addition, the strength and frequency content of the near-field ground motions can also be dramatically different, depending on the orientation of the measurement with respect to the source. Fault-normal and fault-parallel ground motions have been observed to be substantially different in the mid- to long period range (0.5 to 3.0 sec). The influence of these effects on the characteristics of strong ground motions should be incorporated into the design of mid- to long period structures as well as seismic base-isolation systems that are sensitive to large velocities and displacements.

Local Site Conditions The influence of local soil conditions on the strength of the ground shaking has long been recognized as a contributing factor to the geographic distribution of ground failures and structural damage during earthquakes. Of particular concern to earthquake engineers is the amplification of ground motions in period ranges of engineering interest, as well as the progressive softening (nonlinear behavior) of weak soils at high levels of shaking. The extensive collection of recorded strong motion records from earthquakes worldwide (e.g., 1985 Mexico City Earthquake, 1988 Armenian Earthquake, 1989 Loma Prieta Earthquake, 1994 Northridge Earthquake, 1995 Hyogoken Nanbu Earthquake) has contributed to an enhanced understanding of site effects for a wide variety of geologic conditions. These records have been used as the basis for quantitative studies of the influence of soil response on the characteristics of strong ground motions. The effects of site geology on the amplitude of various ground motion parameters such as peak acceleration, velocity and displacement, and also on the frequency content of the motions and their corresponding response spectra, have now been well demonstrated (Borcherdt, 1994; Seed et al., 1994). The potential for significant enhancement of ground motions in any period range is a function of seismological, geologic, and geotechnical factors, several of which are listed in Table 1-1.

In general, the results of seismicity evaluations are presented as the peak acceleration expected for a given return period or as a function of the probability of exceedance with time. In either case, the acceleration usually corresponds to the shaking at a rock outcrop, not at the surface of a soil profile. Site effects must then be evaluated as a function of key parameters such as; soil type, soil thickness, soil stiffness, and the strength of the bedrock

TABLE 1-1:FACTORS INFLUENCING THE MAGNITUDE OF SITE EFFECTS ON STRONG GROUND MOTIONS

SEISMOLOGICAL FACTORS
Intensity of bedrock, or input, shaking
Frequency characteristics of the input motions
Duration of the input motions
GEOLOGIC FACTORS
Soil type(s)
Thickness of the soil deposit
Underlying rock type
Geologic structure
GEOTECHNICAL FACTORS
Low-strain stiffness of the soils (shear wave velocity or maximum shear modulus)
Stiffness (impedance) contrast between the bedrock and overlying soils
Damping characteristics of the soil units
Cyclic modulus degradation behavior of the soils
Relationship between the shear strain and shear stress for predominant soil units
Site period
OTHER FACTORS
Two- and three-dimensional effects (e.g., subsurface bedrock topography, basin effects)

motions. Given this site-specific data, the dynamic response of the soils can be evaluated using either simplified empirical relationships or site-specific dynamic soil response techniques.

With this as background, the following subsections provide an overview of the various procedures available to estimate site-specific ground motions for the seismic design of port structures.

Estimation of Site-Specific Ground Motions

The estimation of site-specific ground motions for engineering design or analysis of port facilities is most typically based on: (a) deterministic or probabilistic methods for estimating site-specific rock motions (usually represented as peak ground acceleration or response spectra); and

(b) modification of these rock motions to account for local soil conditions. In addition, because of the increasing use of nonlinear methods of seismic analysis of port facilities (as discussed in following chapters), it is also often required to develop an appropriate ensemble of ground motion time histories.

Deterministic Methods for Estimating Site-Specific Rock Motions Deterministic methods for estimation of site specific rock motions consist of the following general steps: (a) for each of several potential seismic sources in the vicinity of the port site, estimate the maximum earthquake magnitude associated with that source; and (b) using an appropriate rock motion attenuation relationship, estimate the associated rock motions at the site, as a function of the maximum earthquake magnitude and the distance from the earthquake source to the site. After this is repeated for all potential earthquake sources in the vicinity of the site, the particular set of computed rock motions that lead to the most severe shaking at the site are selected. If a site has several different structures with different natural periods, response spectra that lead to governing motions in the period range of importance for each structure should be selected.

The above deterministic methods for estimation of site-specific rock motions have the advantage of being readily understood by non-technical port personnel and decision makers. However, they represent extreme earthquake scenarios only. Furthermore, they do not account for the uncertainties inherent in the estimation of the size and location of future earthquakes, and the rate at which rock motions attenuate with increasing distance from the seismic source. These factors are best represented using probabilistic methods summarized in the next section.

Probabilistic Methods for Estimating Site-Specific Rock Motions The seismic design or upgrading of a particular port component requires an assessment of the potential level of shaking at the site due to future earthquakes. In much the same way that port and coastal engineers design marine structures in consideration of the largest waves that may occur over the design life of the structure (e.g., 5-, 10-, or 25-year waves), earthquake engineers design for the levels of ground shaking that are anticipated to occur at a particular site with a specific average recurrence interval or return period (e.g., 72- and 475-years, which correspond to probabilities of exceedance of 50% and 10% respectively in 50 years).

This subsection provides an overview of probabilistic seismic hazard analysis (PSHA) methods for estimating site-specific rock motions with a given probability of exceedance or return period. The advantage of these methods is that they account for uncertainties in locations, magnitudes, and recurrence intervals of future earthquakes, and also in the rate of attenuation of rock motions with increasing distance from the seismic source. The seismic hazard analysis results are developed from information that describes the seismicity, geometry, and locations of the significant seismic sources in the region, and appropriate rock motion attenuation relationships for the region. Probabilistic models then synthesize this information to develop the probabilities or recurrence intervals associated with various levels of shaking at the site.

Figure 1-10 provides a flow chart that illustrates the PSHA procedure for developing site-specific uniform hazard spectra (which are spectra whose amplitudes at all natural periods represent the same probability of exceedance). PSHAs using this general approach have been utilized for numerous ports in the United States (e.g., Power et al., 1986; McGuire, 1990) and have formed the basis for the development of regional and national seismic hazard maps

(Geomatrix, 1995; Hanson and Perkins, 1995; Leyendecker et al., 1995; Frankel et al., 1996). However, in the implementation of these methods, the engineer should consider the following factors:

A difficult element of the above probabilistic seismic hazard analysis process is the specification of the rate of seismicity (i.e., earthquake recurrence intervals) in a region. As previously mentioned, the relatively short historic record combined with varying rates of seismicity for the various tectonic provinces in the United States preclude a precise estimate for the recurrence intervals of damaging earthquakes. This uncertainty should be acknowledged and addressed in a straightforward manner. Probabilistic methods are commonly used to identify the uncertainties associated with seismicity rates

The uncertainties in the rate of attenuation of the rock motions with increasing distance from the seismic source is represented in the PSHA either by using an appropriate probability distribution to represent this attenuation rate, or by performing a logic tree type analysis with mean values adjusted to reflect standard deviations in the empirical data.

Figure 1-11 shows the results of PSHA for soft rock-shallow stiff soil sites (NEHRP B-C boundary) in the United States. The parameter being mapped is the peak horizontal acceleration having a 10% probability of exceedance in 50 years. This corresponds to a roughly 475 year recurrence interval, and is equivalent to the exposure time for which the seismic load levels prescribed in current building codes are established. This data demonstrates relative ground shaking hazards for numerous regions in the United States. In light of the fact that seismic design at ports is most commonly based on exposure times of roughly 75 to 475 years,

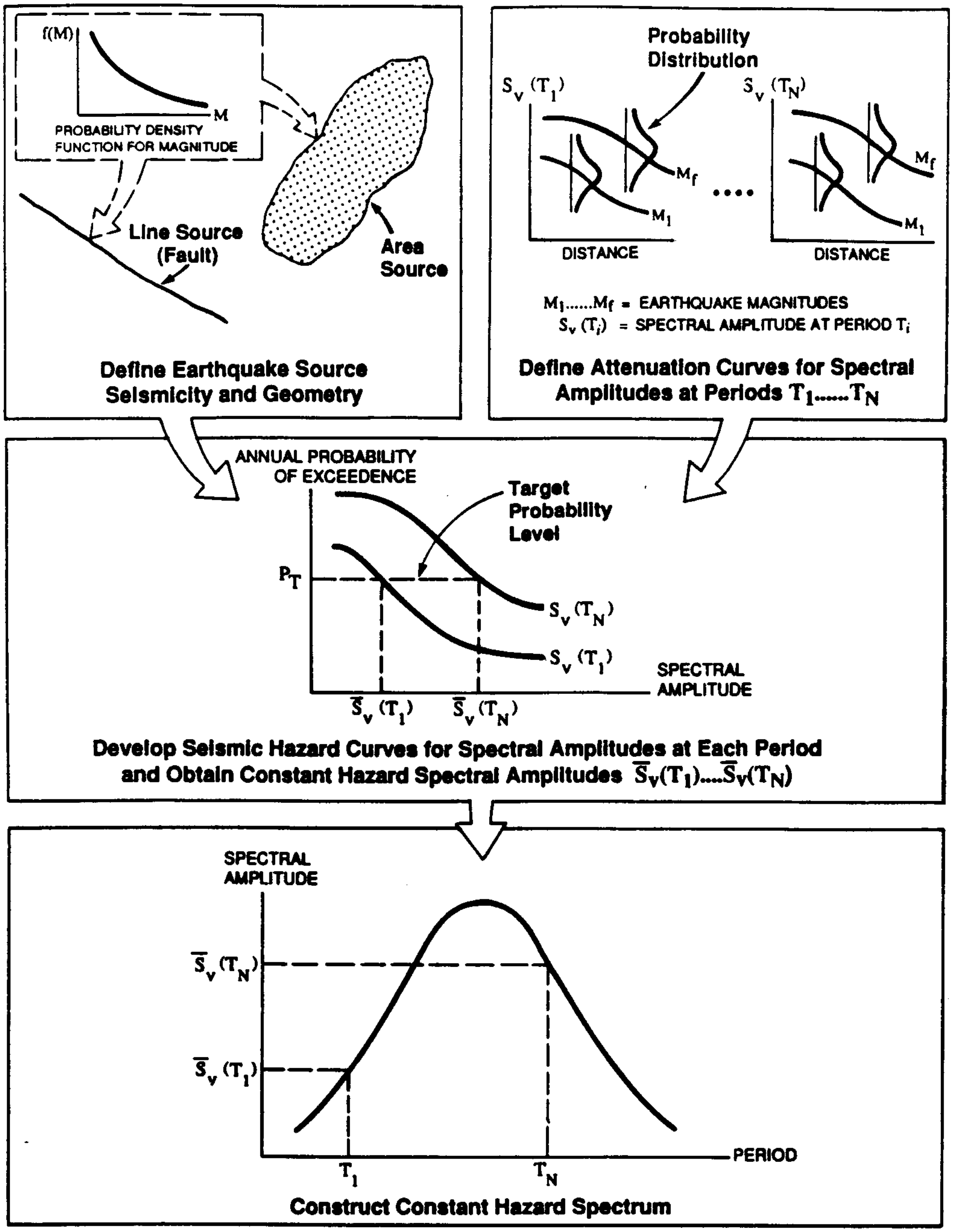


Figure 1-10: Development Of Uniform-Hazard Design Spectra Using Probabilistic Seismic Hazard Analysis Procedures (Werner, 1991)

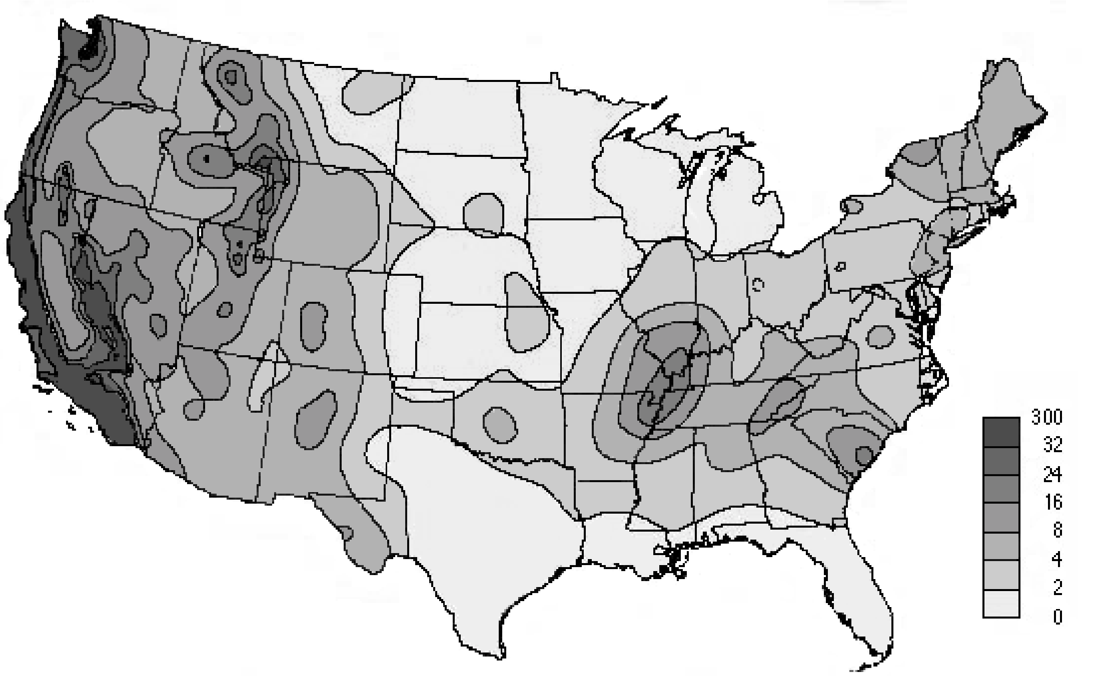


Figure 1-11: Peak Acceleration (%G) With 10% Probability Of Exceedance In 50 Years (Frankel Et Al., 1997)

Note that this map can be generated for all, or portions, of California.
The maps are available at the USGS seismic hazards and mapping web-site

it is evident that many regions of interest are considered prone to ground motions on rock that approach, or exceed, 0.10 g. Once the PSHA has been completed, maps such as this can be generated for a variety of ground motion parameters (e.g., peak ground motions, spectral accelerations at specified periods) and exposure times.

PSHA demonstrates the effect that the return period (or exposure time) has on the strength of ground motions anticipated at a specific site. The recurrence interval selected for the design of port facilities is a function of the seismic risk that can be accepted by the port authority. The variation in the peak ground acceleration having a 10% probability of exceedance is shown as a function of the exposure time in Figure 1-12. The data for this figure has been compiled from NEHRP (1993), Frankel et al. (1997) and Cox and Chock (1991). The results of probabilistic analyses such as these can be used by port engineers to assess the influence of exposure time on the seismic hazard.

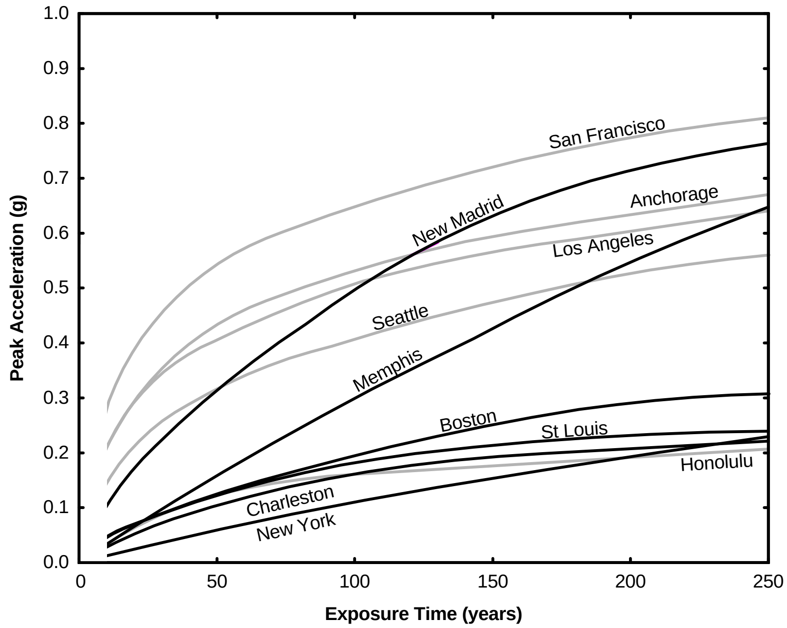


Figure 1-12: Peak Accelerations For U.S. Sites Assuming NEHRP B-C Rock Site Conditions (Motions Having A 10% Probability Of Exceedance During The Selected Exposure Time)

It is common for port engineers to use a performance-based criteria in seismic resistant design that requires the definition of two levels of ground motion for the design and analysis of structures. As an example, the guiding principles used for specifying the earthquake motions used in seismic design at the Port of Los Angeles (POLA) are as follows (Wittkop, 1997):

> Moderate earthquake motions (defined as Operating Level Earthquake motions, or Level 1 earthquake motions) should be resisted by wharf structures, retaining structure/dikes and critical operational structures and facilities founded on the backland fill areas, with only minor non-structural damage. From a design standpoint, deformations of wharf structures should not result in significant residual cracking or spalling of the concrete or permanent elongation of the steel reinforcement, and deformations of critical operational structures and facilities should remain within the elastic range. In their seismic design criteria, POLA defined the Operating Level Earthquake (OLE) motions as having a 50% probability of exceedance in 50 years (which is roughly a 72 year recurrence interval)
>
> Large earthquake motions (designated by POLA as Contingency Level Earthquake motions) should be resisted by wharf structures, retaining structure/dikes and critical operational structures and facilities in a manner which prevents collapse and major structural damage. Damage that does occur should be readily detectable and accessible for inspection and repair. Design concepts should be such that damage to foundation elements below ground level should be prevented. Container cranes and critical operational structures and facilities should remain operable with only minor repairs. The Contingency Level Earthquake (CLE, or Level 2) motions have been defined by POLA as having a 10% probability of exceedance in 50 years (or a 475 year return period). This is equivalent to the exposure time for ground

motions used in the development of building codes.

It should be noted that this is just one example of performance-based seismic design criteria. Other ports may establish specific acceptable performance guidelines for different components based on the importance of the facility. Also, the exposure times selected by the Port of Los Angeles reflect the regional rate of seismicity. In the Los Angeles area, the Level 1 ground motions correspond to moderate levels of shaking that are likely to occur at least once during the life of the structure. Level 2 ground motions are much more severe levels of shaking that have a more remote potential for occurrence at the site during the life of the structure. For similar structures and construction practices, the exposure times adopted for use in seismic design will vary from region to region due to the variation in seismicity rates.

As noted previously, it has been common practice throughout much of the United States to use probabilities of exceedance of 50% in 50 years and 10% in 50 years to represent Level 1 and Level 2 ground motions for seismic design and analysis. However, this does not provide uniform protection across the state (e.g., Central Valley). To illustrate this across the United States, the data presented in Figure 1-12 has been replotted in Figure 1-13 in terms of the peak ground acceleration having a 10% probability of exceedance in a specified time interval divided by the peak ground acceleration having a 10% probability of exceedance in 50 years versus exposure time. These results show that for the western United States cities shown in Figure 1-13, the use of a probability level of 10% in 50 years to represent the Level 2 ground motions provides a reasonable estimation of the near-maximum levels of ground shaking that can occur over much longer exposure times. However, this is not true for sites in the central and eastern United States. For these regions, Figure 1-13 shows that the use of a probability level of 10% in 50 years may not provide adequate protection against the much larger near-maximum levels of ground shaking that can be associated with longer exposure times in these regions. The establishment of Level 2 ground motions for seismic design or retrofit of critical facilities at port in the central and eastern United States should carefully consider this trend, and may warrant the use of Level 2 ground motions with much longer exposure times for such ports. As discussed in other chapters, these considerations are now reflected in the new (1997) NEHRP seismic design provisions for buildings.

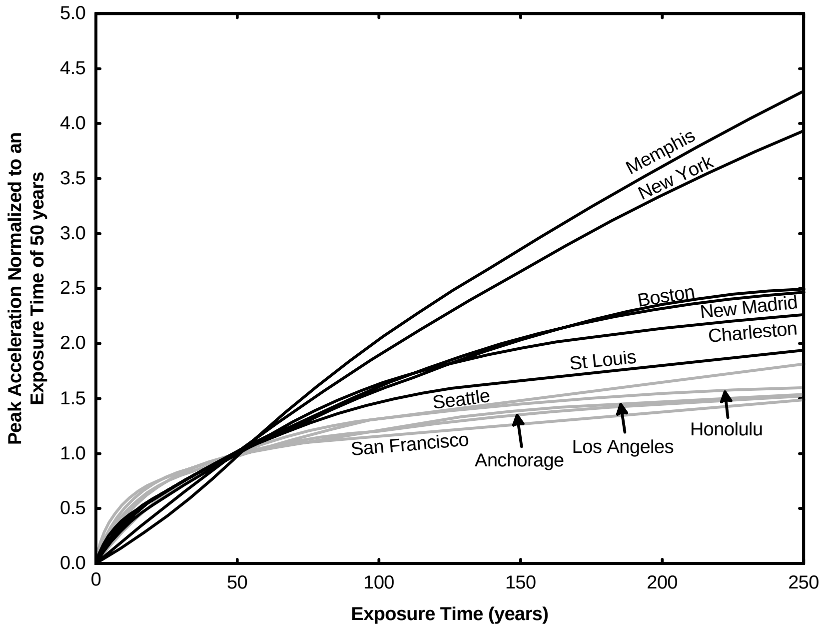


Figure 1-13: Normalized Peak Accelerations For U.S. Sites Assuming NEHRP B-C Rock Site Conditions

Local Soil Effects on Ground Surface Motions

Background Seismic waves which propagate from the underlying rock into near surface soil deposits are modified by the dynamic response characteristics of the local soils. The influence of the soil deposit on the bedrock motions will depend on the characteristics of the input motions, the thickness of the soil deposit, and the dynamic behavior of the individual soil layers. This aspect of the seismic hazard evaluation focuses on the dynamic response of soil deposits, or *site effects.*

In the last 15 years, the extensive collection of recorded strong motion records from worldwide earthquakes has contributed to an enhanced understanding of site effects for a wide variety of geologic conditions. The effects of site geology on the amplitude of ground motion parameters such as peak acceleration, velocity, and displacement, as well as the frequency content of the motions and their corresponding response spectra has been well demonstrated. An example from the U.S. Navy facility at Treasure Island during the 1989 Loma Prieta, Figure 1-14. The influence of the local soils on the characteristics of the ground motions is apparent. In addition to amplifying the peak ground acceleration, the dynamic response of the soil has resulted in enhanced motions at all periods between 0 and 4 seconds (as demonstrated by the response spectra).

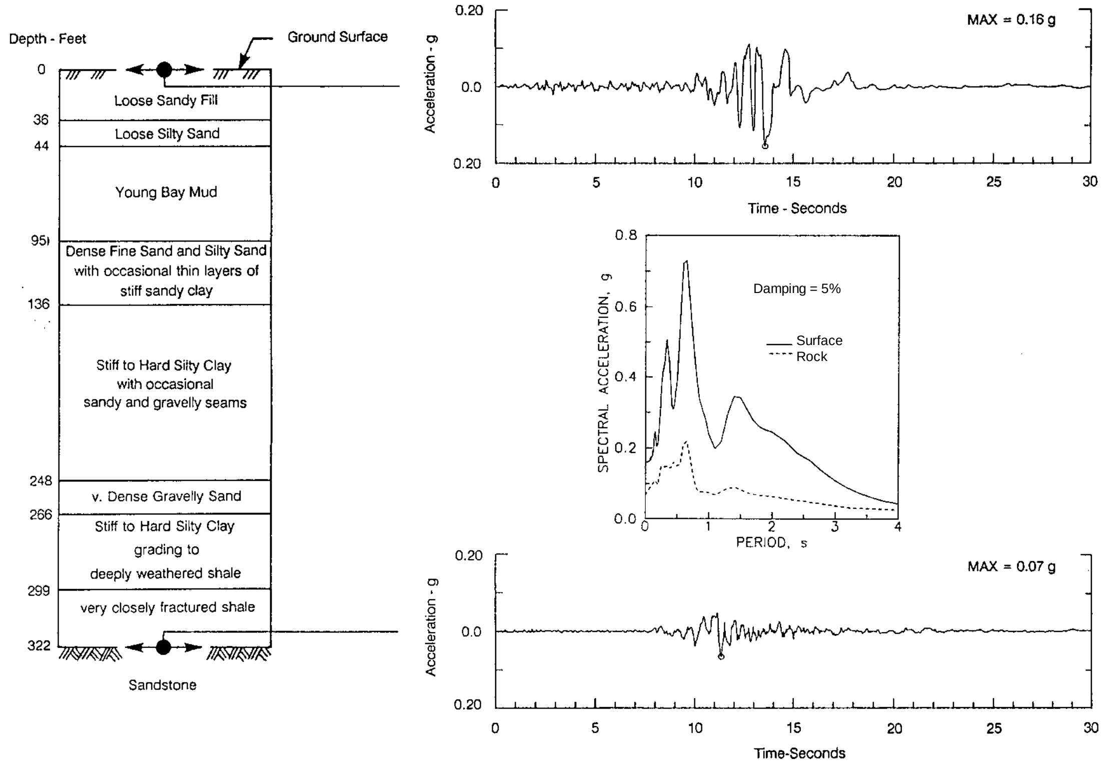


Figure 1-14: Soil Response At Treasure Island During The 1989 Loma Prieta Earthquake (After Seed Et Al., 1990)

Site effects can lead to enhanced ground motions at intermediate to longer periods of vibration, the range of concern for many structures. Spectral ratios (spectral acceleration of the ground surface motion divided by the spectral acceleration for the rock motion at the corresponding period) are commonly used to highlight the influence of the soil deposits on the characteristics of the strong ground motions. The spectral ratios for sites affected by the 1985 Mexico City Earthquake and the 1989 Loma Prieta Earthquake are shown in Figure 1-15. In both cases, the ground motions at intermediate periods (1 to 4 seconds) have been substantially amplified by the clayey soil deposits at the 15 sites documented. The relative amplification ratios are primarily functions of the stiffness of the clayey soils. The Mexico City clays are considerably less stiff than the San Francisco Bay muds.

Empirical studies of the effects of dynamic soil response on the characteristics of rock motions have been well documented in the geotechnical and seismological literature (e.g., Seed and Idriss, 1982; Borcherdt, 1994; Seed et al., 1994). These investigations have focused on two primary aspects of site response: (a) amplification of the peak acceleration on rock; and (b) amplification of spectral accelerations computed for the rock motions. Site soil effects on rock accelerations have been demonstrated in plots of PGA_{soil} versus PGA_{rock}, Figure 1-16. Given the peak acceleration for rock, the corresponding peak acceleration at the ground surface can be easily estimated. Similar plots have been developed for estimating spectral amplification ratios as well. In the aftermath of the 1989 Loma Prieta Earthquake, substantial research effort on this topic has led to the development of simple, yet suitably precise, techniques for developing acceleration response spectra at soil sites. The methodology that has been adopted for use in current seismic design codes is presented below.

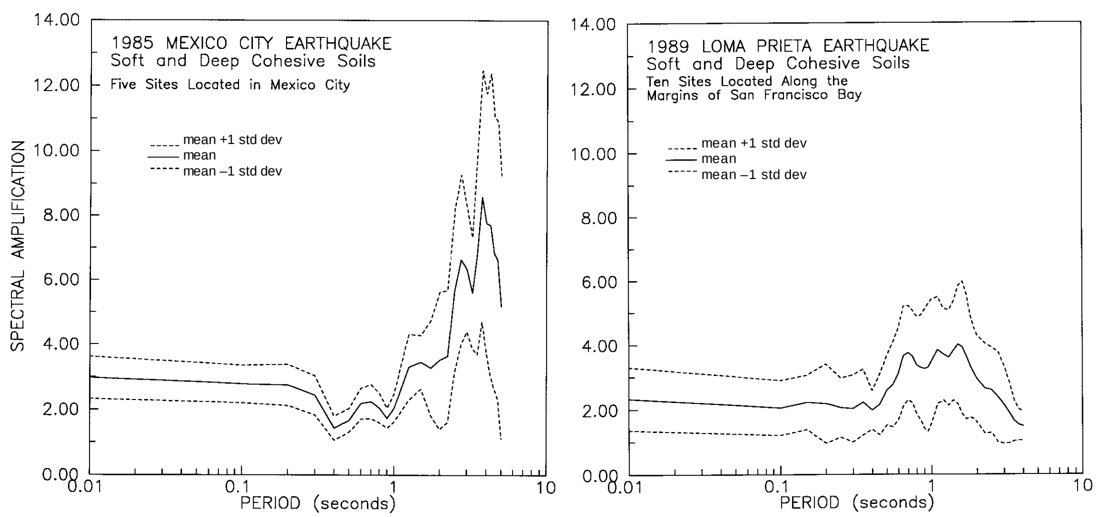


Figure 1-15: Spectral Amplification At Soft And Deep Cohesive Soil Sites (Dickenson And Seed, 1996)

Specification of Site Effects in Current Seismic Design Provisions The specifications that govern site effects in current building codes have been developed and adopted by a number of governmental agencies and engineering organizations over the past two decades. This summary focuses on the techniques for incorporating dynamic soil response in two current codes and recommended seismic design provisions (ICBO 1997; FEMA, 1998). This brief overview of the seismic design provisions is intended to highlight the strengths and limitations of these methods for use in practice.

The combination of the strong motion records obtained during the 1989 Loma Prieta Earthquake and extensive site characterization at strong motion instrument stations made possible with various in situ testing techniques (e.g., SPT, CPT, shear wave velocities) has provided the means for developing enhanced site classes for use in seismic design codes.

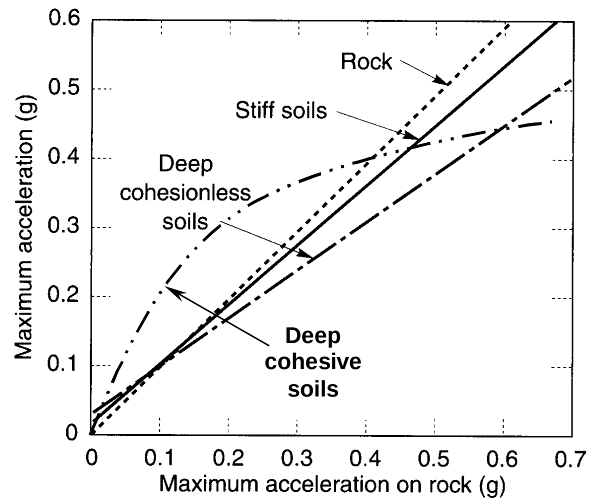


Figure 1-16: Approximate Relationships Between Peak Accelerations On Rock And Other Local Site Conditions (After Seed And Idriss, 1982; Idriss, 1990)

Comprehensive studies of recorded ground motions obtained for a wide variety of source characteristics and site conditions, and dynamic soil response analyses for a wide range of soil conditions have also been utilized to establish the expanded set of site classes that have been incorporated into current site coefficients and acceleration response spectra.

The new National Earthquake Hazard Reduction Program (NEHRP) and Uniform Building Code (UBC) provisions for site effects provide six well-defined site classes, as well as amplification factors that depend both on site conditions and on the level of the site-specific rock accelerations. The recommended site categories are specified in terms of the stiffness and strength of the upper 30 meters (100 feet) of the soil profile, Table 1-2. Exceptions to the 30 meters depth used for classification proposes are made for soil profiles that include very weak, metastable soils (site classes E and F). In these cases, thin near-surface layers can result in severe damage to foundations and retaining structures.

In addition to the incorporation of a more well defined site classification system, one of the primary improvements in the new seismic design provisions is the utilization of intensity-dependent amplification factors for modifying short- and intermediate period rock motions. The basis for the seismic hazard evaluation in current versions of the NEHRP and UBC seismic design provisions are spectral accelerations at selected response periods, and effective peak accelerations for rock sites, respectively. The respective ground motion parameters are obtained from maps then multiplied by site coefficients applicable to short-period motions and mid-period motions. These methods have purposely been developed so that it can easily be used with other site- or region-specific spectral maps which may be developed (e.g., Martin and Dobry, 1994; Geomatrix, 1995; Leyendecker, et al., 1995). The amplification factors, or *Site Coefficients*, provided in Tables 4-3 and 4-4 reflect both site effects at the different period ranges and the nonlinear behavior of soils. Given the site classification and values of A_a and A_v, the soil amplification factors can be determined.

TABLE 1-2:SITE CLASSIFICATIONS FOR USE IN SEISMIC DESIGN CRITERIA

SOIL PROFILE TYPE	GENERAL DESCRIPTION	SHEAR WAVE VELOCITY (m/sec)	STANDARD PENETRATION RESISTANCE (blows/30 cm)	UNDRAINED SHEAR STRENGTH (kPa)
A	Hard rock	> 1,524	n/a	n/a
B	Rock	762 to 1,524	n/a	n/a
C	Very dense soil and soft rock	366 to 762	> 50	> 96
D	Stiff soil	183 to 366	15 to 50	48 to 96
E	Soil profile with $V_s < 183$ m/sec, or any profile with more than 3 m of soft clay defined as soil with plasticity index > 20, water content > 40%, and undrained shear strength < 24 kPa.			
F	Soils requiring site-specific evaluations*.			

This straightforward technique provides a useful estimation of site specific soil response. The methodology involves the following steps;

a) Determine the "design-level" peak horizontal acceleration in bedrock (A_a) from: (a) available seismic zone maps; (b) an appropriate attenuation relationship; or (c) by means of site-specific seismicity studies.

b) Select a representative site category from Table 1-2. The site class is determined by obtaining an average shear wave velocity for the upper 30 meters of soil. The shear wave velocities are either measured using geophysical techniques, local shear wave velocity data in the same geologic units, or estimated from established correlations with other geotechnical properties (e.g., SPT or CPT penetration resistance, void ratio, undrained shear strength) for each of the foundation soils.

c) Select the short- and mid-period amplification factors (F_a, F_v) Table 1-3 and Table 1-4.

d) Compute spectral accelerations (S_A) at short periods using the formula $S_A = 2.5 \cdot F_a \cdot A_a$, and compute spectral accelerations at mid-to-long periods using the expression $S_A = F_v \cdot (A_v/T)$, where T is the period in seconds. Then, plot the elastic acceleration response spectrum (5% damping) as shown in Figure 1-17.

TABLE 1-3: VALUES OF Fa AS A FUNCTION OF SITE CONDITIONS AND SHAKING INTENSITY

SOIL PROFILE TYPE	SHAKING INTENSITY				
	$A_a < 0.1$	$A_a = 0.2$	$A_a = 0.3$	$A_a = 0.4$	$A_a > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	b
F	b	b	b	b	b

Notes:
a. Use straight line interpolation for intermediate values of A_a.
b. Site specific geotechnical investigation and dynamic site response analyses shall be performed.

TABLE 1-4. VALUES OF Fv AS A FUNCTION OF SITE CONDITIONS AND SHAKING INTENSITY

SOIL PROFILE TYPE	SHAKING INTENSITY				
	$A_v < 0.1$	$A_v = 0.2$	$A_v = 0.3$	$A_v = 0.4$	$A_v > 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	b
F	b	b	b	b	b

Notes:
a. Use straight line interpolation for intermediate values of A_v.
b. Site specific geotechnical investigation and dynamic site response analyses shall be performed.

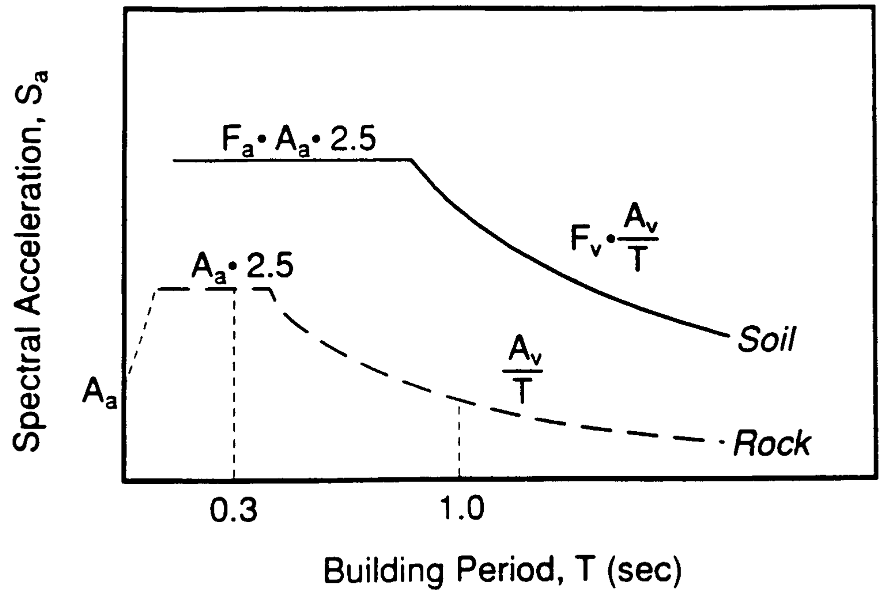


Figure 1-17: Two-Factor Approach To Defining Design Spectra (FEMA, 1995)

Numerical Ground Response Analysis Methods for Evaluating Site Effects

Despite recent improvements in the methods for constructing soil-dependent acceleration response spectra now contained in the NEHRP and UBC seismic design provisions, aspects of the soil profile or structure under consideration may warrant a site-specific response analysis. In certain instances, the seismic design provisions in building codes prescribe that the simplified methods of evaluating dynamic soil response should be augmented with the results of site-specific response analyses. Seismic design which accounts for near-source effects, soft or potentially unstable soils, critical structures, or structures with plan-irregularities, may require more rigorous response analyses than are outlined in building codes.

The engineer has at his or her disposal a variety of computer programs that can be used to predict the dynamic response of soil deposits. The level of sophistication of these numerical methods (and the soil data and engineering time required) varies considerably with the more complex programs requiring as many as 20 soil parameters for each soil layer in the model. In addition, the computer programs that have been developed for modeling dynamic soil response rely on various simplifications and assumptions in order to solve equations for wave propagation through soils. The spectrum of computer-based analyses for dynamic soil response ranges from relatively simple linear-elastic total stress soil models to sophisticated and fully nonlinear effective stress techniques. A cursory introduction to dynamic soil models is provided, followed by several practical insights on the performance of analytical soil response programs.

The influence of the soil deposit on the amplitude of the incident seismic waves will be greatest near the predominant period of the deposit. This period can be estimated for the case of vertically propagating waves in a linear elastic soil media from the relation:

$$T = \frac{4H}{(V_s)_{AVG}}$$

where H is the depth of the deposit and $(V_s)_{AVG}$ is the average shear wave velocity of the deposit. While this simple relationship provides a useful insight into the period range at which site effects may be significant, it does not address the magnitude of this amplification on the ground motions. This amplification is a function of the thickness and stiffness of the soils, the contrast between the stiffness of the soil and underlying rock (*impedance contrast*), and the strain-dependent properties of the soil. The response of a multi-layered soil profile subjected to transient motions is a complex phenomenon which involves strain- and frequency-dependent behavior, hysteretic stress versus strain soil properties, and potential fatigue related phenomena such as modulus degradation and excess pore pressure generation. This behavior is clearly nonlinear and difficult to model analytically. In order to account for these and other factors in the analysis of dynamic soil response, computer programs must be employed.

Requisite input parameters and modeling details for these analyses of seismic soil response include: (a) suitable strong motion records (digitized acceleration time histories); and (b) representative dynamic properties for soils at the site. In addition to the unit weight (γ_t), which can be readily estimated, the two principal dynamic soil properties of interest in response analyses are: (a) the dynamic shear modulus, G, which describes the stiffness of the soil; and (b) some measure of dynamic material damping (i.e., the damping ratio, β, which is related to the energy lost per cycle of shaking).

The shear wave velocity is a useful parameter for describing the small-strain (cyclic shear strains $\leq$ 1.0 x 10^{-4}%), and the corresponding maximum stiffness of the soil. The shear wave velocity can be measured in situ using common testing techniques, and it is now used as one of the criteria for classifying soil deposits in seismic design codes, Table 1-2. The shear wave velocity is related to the dynamic shear modulus of the soil by the simple formula:

$$G_{\max} = \frac{V_s^2 \gamma_t}{g}$$

where G_{max} is the small strain dynamic shear modulus, V_s the shear wave velocity, γ_t the total unit weight of the soil, and g is the acceleration of gravity. Despite recent advances in sampling and laboratory testing techniques, the adverse effects of unavoidable sample disturbance on the small strain dynamic modulus of a soil, as well as difficulties associated with small-strain measurements, render in-situ seismic wave velocity measurements the currently preferred method for determining G_{max}.

Both dynamic shear modulus and damping are “nonlinear” properties of soils: both are strongly dependent on shear strain levels. As shear strains increase the dynamic moduli decrease and damping increases as shown in Figure 1-18. Computer programs that are most commonly used in engineering practice for the development of a site-specific acceleration response spectrum are based on the assumption of vertically propagating seismic waves through horizontally layered soil deposits. These simplifications allow the response analysis to be performed on the basis of one-dimensional (1-D) wave propagation. For most engineering work, the assumption of vertically propagating waves is not unreasonable, due to the refraction of waves at layer interfaces as the waves travel from deep, dense material upward through soils which are progressively less-dense and subjected to reduced confining stresses. The limitations imposed by 1-D analyses include several effects that can influence site-specific ground motions. Among these effects are two-dimensional and three-dimensional (2-D and 3-D) bedrock topography, basin effects, wave-scattering, horizontally propagating surface waves, and sloping ground conditions.

Equivalent Linear Dynamic Soil Response Method The most commonly used equivalent linear soil response model is incorporated in the program SHAKE which was originally developed by Schnabel et al. (1972) and later updated by numerous individuals (e.g., Idriss and Sun, 1992). The SHAKE program employs an equivalent linear total stress analysis to compute the response of a horizontally layered visco-elastic system subjected to vertically propagating shear waves. In this, an exact continuum solution ("shear-beam") to the wave equation is adapted for use with transient motions, through the Fast Fourier Transform (FFT) algorithm. The FFT essentially replaces the transient motion represented by the digitized acceleration time history by a finite series of harmonic motions. The hysteretic stress-strain behavior of soils under symmetrical cyclic loading is represented by an equivalent modulus, G, corresponding to the secant modulus through the end points of the hysteresis loop and an equivalent damping ratio, □, corresponding to the equivalent damping. The equivalent modulus and damping ratio are equivalent-linear, strain-dependent properties. Modulus reduction and damping curves (such as those shown in Figure 1-18) are incorporated into input files to model the nonlinear dynamic properties of the soils.

The shear moduli and damping corresponding to the computed shear strains are determined using an iterative procedure that is based on linear dynamic analysis. For all soil sublayers estimates of the dynamic moduli (G_{max} or V_s) and a damping ratio ($\beta \approx 1 - 5\%$)

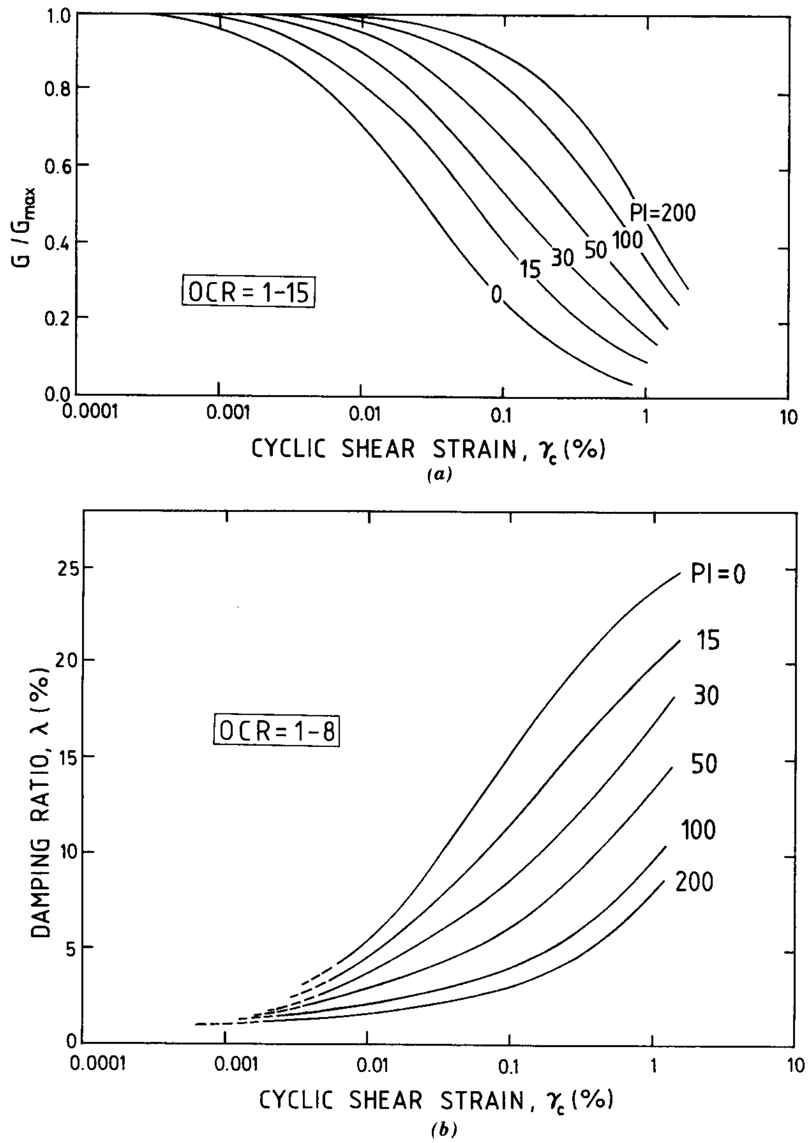


Figure 1-18: Relationships Between The Shear Strain And The Shear Modulus And Damping Ratio For Normally And Overconsolidated Soils (Vucetic And Dobry, 1991)

are provided for the first iteration. The equivalent linear method incorporated in SHAKE then approximates nonlinear soil behavior with an iterative method that uses a linear wave propagation formulation with soil properties that are compatible with equivalent uniform, or "effective", shear strain levels which are assumed to exist within each of the soil sublayers for the duration of the excitation. The ratio of this equivalent uniform shear strain to the calculated maximum strain is specified as an input parameter (**n**) and the same value of this ratio is used for all sublayers. The ratio **n** is computed using the simple formula proposed by Idriss (Idriss and Sun, 1992); **n** = (**M**-1)/10 where **M** is the moment magnitude of the earthquake being modeled. At each iteration, **n**% of the peak strains computed at the mid-point of each soil sublayer from the previous iteration are used to obtain new values of strain-dependent modulus and damping ratio. The program iterates until the modeled strain-dependent soil properties are compatible with the strain levels associated with the calculated response of the system. The final computed soil properties are thus referred to as “strain-compatible” properties

The widespread use of SHAKE, and other similar equivalent linear methods in engineering practice is due to its relative ease of use and the limited number of input parameters required for each soil layer. In addition to the digitized acceleration time history and the strain-dependent modulus and damping curves, each soil layer is completely described its thickness, unit weight, low-strain modulus and damping. The simplicity of the SHAKE model results in an economy of effort when preparing input files and interpreting computer-generated output.

Although the equivalent linear method of analysis has been found to provide acceptable results for many engineering applications, a number of limitations have been noted in the technical literature. In addition to the inherent limitations of 1-D response analyses previously noted, practicing engineers should be aware of the following issues when performing equivalent linear dynamic response analyses:

The equivalent linear model is based on a linear elastic formulation. Because all hysteresis loops are symmetric about the origin, permanent (plastic) deformations are not modeled.

Once the strain-compatible soil properties have been obtained for each layer, single values of G and □ are used throughout the final analysis. Moduli and damping ratios are independent of frequency. In addition, the soil properties associated with the largest strains are slightly “under-softened” and under-damped, while those at lower strains are over-softened and over-damped.

The equivalent linear method employed in SHAKE performs a total stress analysis. Although the output (uniform stresses or strains) can be used in a decoupled analysis of excess pore pressure generation, the program does not perform an analysis wherein the stiffness of the soil is modified at each iteration to account for generated excess pore pressures. The excess pore pressures that can be generated in loose to medium dense sands and sensitive cohesive soils are not accounted for. These excess pore pressures result in progressive softening of the soil, a reduction in the high frequency components of the motions, and potentially large permanent displacements.

No upper limit is placed on the peak equivalent uniform shear stress that is computed in each layer. In cases where moderate to strong levels of shaking are input at the base of a soil profile which includes soft to medium stiff cohesive soils the computed shear stresses often exceed the dynamic shear strength of the soil. The result is overprediction of peak ground accelerations and high frequency motions. The strain dependent soil properties can be modified to reduce the computed stresses.

Despite these limitations, numerous validation studies have demonstrated the accuracy of the 1-D, equivalent linear method to model the dynamic response of various soil profiles for which ground surface and representative rock input motions are available (Seed et al., 1994; Idriss, 1993a).

Fully Nonlinear Dynamic Soil Response Methods In order to overcome the deficiencies of total stress equivalent linear soil response methods, fully nonlinear effective stress analyses have been formulated. These numerical methods are based on a time-domain solution wherein the response of the soil is evaluated in a stepwise manner at each point in the acceleration time history. The fully nonlinear methods offer a number of improvements in the computation of dynamic soil response in that the following effects can be accounted for: (a) strain- and frequency-dependent soil properties; (b) more accurate modeling of the stress-strain response of the soil; (c) specification of peak undrained shear strengths; and (d) the generation and dissipation of excess pore pressures can be included in the analysis.

In order to model the dynamic behavior of the soil during loading, unloading, and reloading, additional soil parameters are required for each layer. From a practical standpoint, the application of these models is often precluded by the cost of obtaining representative soil properties required as input information. This occasionally leads to the reliance on default values that may be supported by relatively few laboratory investigations or well-documented case histories.

Among the more commonly used 1-D nonlinear programs are DESRA-2C (Lee and Finn, 1991) and SUMDES (Li et al., 1992). Although a complete description of the constitutive models incorporated into these respective programs is beyond the scope of this summary, both of these programs are capable of performing total stress, as well as coupled effective stress analyses. The constitutive relationships utilized in DESRA-2C for an effective stress dynamic response analysis with redistribution and dissipation of porewater pressure require 18 material constants for each soil layer. The hyperbolic stress-strain relationship is used and these soil parameters describe the unit weight, maximum shear strength, maximum shear modulus, stress-strain behavior, volumetric strains, material hardening, permeability, and viscous damping. The computer program SUMDES incorporates a sophisticated plasticity model (termed the *bounding surface hypoplasticity model*) that is based on critical state soil mechanics. The program uses a multi-directional formulation and plasticity models for soil behavior that facilitate the modeling of shear waves and compression waves simultaneously. With this technique, horizontal motions, vertical settlement, shaking induced lateral stress variations, soil compression and dilation, liquefaction behavior of sandy soils, and rotational shear can be modeled. Five levels of analysis are available with the two most complex models requiring 19 to 20 soil parameters for each layer.

The dynamic modeling capabilities of these programs clearly exceeds that provided by the equivalent linear methods. Potentially important dynamic soil behavior such as progressive softening due to the generation of excess pore pressures, limiting shear strength and permanent soil deformations can be evaluated with these fully nonlinear soil models. However, in practice, the advantages provided by the fully nonlinear soil response programs must be weighed against the cost of laboratory testing programs required to obtain representative soil properties for the analyses, as well as the engineering time necessary for the development of input files, performance of parametric studies, and additional scrutiny of the analytical results. The engineer must balance economy of use with usefulness of the output. For example, a response analysis of a shallow, stiff clay deposit under moderate levels of shaking will likely not warrant a fully nonlinear effective stress analysis. Conversely, the characterization of the dynamic response of an extensive deposit of medium dense saturated sandy soil subjected to similar levels of shaking may require a coupled effective stress analysis. In either case the results of these analyses should always be tempered with sound engineering judgment.

Ground Motion Time Histories

Requisite input for numerical seismic analyses of soil deposits and/or port facilities include time-history representations of the site-specific ground motions. Digitized accelerograms (i.e. acceleration time histories) are the most common form of seismic input employed in numerical models. These accelerograms should be consistent with the design spectra developed for the site, and should represent the anticipates shaking at the site due to all of the significant potential earthquake sources in the vicinity of the site. The development of ground motion time histories for numerical analyses, as well as for general seismic design applications, has been based primarily on measured and processed accelerograms contained in the current strong motion database. In addition, various types of synthetic accelerograms have been used for certain applications.

Strong Motion Records A key source of motion-time histories for seismic design purposes is the strong motion data base, which contains an extensive array of recorded and processed accelerograms. Agencies such as the U.S. Geological Survey, NOAA National Geophysical Data Center, and the California Division of Mines and Geology distribute digitized strong motion records that can be used for seismic analyses and design. The various stations at which these accelerograms were recorded represents a wide variety of geologic, tectonic, and subsurface soil conditions -- all of which can influence the characteristics of the recorded motions. Differences in the conditions at the instrument locations for the various accelerograms are undoubtedly the source of the marked differences in the features of these recorded motions. Therefore, the engineer must be aware of the significance of these conditions, and should judiciously select an ensemble of accelerograms that collectively best represent the particular conditions at the project site. In addition, such accelerograms may be adjusted, where appropriate, such that; (a) the composite spectra from the accelerograms are reasonably consistent with the design spectra developed for the structures; and (b) the composite characteristics of the accelerograms are reasonably consistent with those indicated by applicable ground motion attenuation relationships. In many regions of the United States, the lack of a robust collection of strong motion recordings precludes the acquisition of a representative ensemble of natural time histories for seismic design purposes. In addition, site-specific aspects of a particular project (i.e., location relative to the seismic source, local geography and geologic setting) may eliminate the existing record from consideration even in well-instrumented, seismically active portions of the country. In cases such as these, synthetic earthquake ground motions can be generated.

Synthetic Accelerograms The development of synthetic accelerograms was first motivated by the need to partially fill important gaps in the current strong motion database. Toward this end, various methods based on random vibration theory or on analytical wave propagation models were used to develop synthetic accelerograms that embody the basic characteristics of strong motion records, as indicated by available data and by engineering judgement regarding future earthquakes (Silva and Lee, 1987).

In practice, it is common for the design ground motions to be described in terms of a peak acceleration and a specified target spectra or design response spectra. The design response spectra may be established using empirical equations as described above. The spectral shapes obtained using these equations are generally smooth and somewhat "broad band" in that they do not exhibit the peaks and valleys characteristic of the response spectra computed from natural ground motions. The corresponding synthetic motions are then produced so that their response spectra closely matches the smooth design response spectra.

It should be noted that synthetic ground motions developed in this manner are often more robust than actual ground motions, due to the fact that they are constructed to match a broad, smooth spectrum and therefore contain significant seismic energy at all frequency contents. As a result, such synthetic accelerograms may not be appropriate for use in analyzing the nonlinear response of structures or soil deposits, which will be strongly dependent on the signature of the input motions. Therefore, care and judgment should be exercised when identifying suitable spectrum-compatible accelerograms for use in nonlinear structural analyses, and when interpreting the results of such analyses.

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